تصميم الخزانات العالية والمسنودة على التربة

الطريقة الديناميكية بتحليل اطوار طيف الاستجابة

امثلة محلولة وفق الكود المصري والأوربي

نماذج تصيم الخزانات العالية والأرضية

المسقوفة والمكشوفة - مادة الخرسانة او معدنية

1- خزان عالي خرسانة مسلحة مسقوف محمول على اطار (4) اعمدة

2- - خزان عالي خرسانة مسلحة مسقوف محمول على اطار (6) اعمدة

3 - خزان عالي خرسانة مسلحة مسقوف محمول على كور خرسانة مسلحة

4 – خزان ارضي معدني دائري مسقوف مسنود مباشرة على التربة

5 – خزان ارضي مكشوف خرسانة مسلحة دائري مسنود مباشرة على التربة

6- – خزان ارضي مكشوف خرسانة مسلحة مستطيل مسنود مباشرة على التربة

مقدمة

- من المعروف ان خزانات المياه والسوائل تمتاز بقدرة قليلة في استيعاب وتشتيت طاقة الزلزال واقل مطاوعة وممطولية بالمقارنة مع الأبنية التقليدية طبقا لما جاء في كودات الزلازل حول الأبنية وخزانات المياه والسوائل حيث ثوابت وفرضيات التصميم الزلزالي في الخزانات تعطى قوى قص قاعدي اكبر تقريبا بسبع مرات من الأبنية

- السلامة الزلزالية ذات اهمية كبيرة لصهاريج تخزين مياه الشرب و السوائل

- يجب أن تبقى خزانات المياه تودي وظيفتها في فترة الزلزال وضمان إمدادات المياه الصالحة للشرب إلى المناطق المتضررة من الزلزال وتلبية الحاجة لمكافحة الحرائق. قد يحتوي السائل في الخزاناتسوائل سامة والقابلة للاشتعال وهذه الخزانات يجب أن لا تفقد محتوياتها أثناء الزلزال.

- خزانات السوائل هي أساسا من نوع :

- الأرضي - والمرتفع عن الأرض - والعالية

- هي تستخدم أساسا كخزانات مياه الشرب في الأبنية العالية

- وتكون اما محمولة على اعمدة او كور اسطواني من جدران خرسانة او مسنودة مباشرة على الأرض التربة

- ويمكن تصميمها واشادتها من مواد الخرسانة المسلحة او عناصر معدنية فولاذية

SEISMIC DESIGN OF LIQUID STORAGE TANKS

Design elevated tanks Dynamic Analysis

***Response spectrum** Spring mass model*

Examples solved according to

Egyptian - European -Code

Models High tanks – floor-design

Roofed - exposed - concrete - metal

1. High-tank reinforced concrete roofed propped on a framework (4) columns

2. - High-roofed reinforced concrete tank propped on a framework (6) columns

3 - High-roofed reinforced concrete tank propped on Core Reinforced Concrete

4 - Ground-roofed metal circular tank propped directly on the soil

5 - Ground tank Exposed reinforced concrete circular propped directly on the soil

6. - Ground tank Exposed reinforced concrete rectangular propped directly on the soil

. INTRODUCTION

-It is well recognized that liquid storage tanks possess low ductility and energy Absorbing capacity as compared to the conventional buildings. Accordingly, various Design codes provide higher level of design seismic forces for tanks. In this article, Provisions of IBC 2000, ACI, AWWA, API, Euro code 8 and NZSEE guidelines are Reviewed to assess the severity of design seismic forces for tanks vis-à-vis those Buildings It is seen that, depending on the type of tank, design seismic force for tanks Seismic safety of liquid storage tanks is of considerable importance.

Water storage tanks should remain functional in the post earthquake period to ensure potable water supply to earthquake-affected regions and to cater the need for fire fighting. Industrial liquid containing tanks may contain highly toxic and inflammable liquids and these tanks should not loose their contents during the earthquake. Liquid storage tanks are mainly of two types: ground supported tanks and elevated tanks. Elevated tanks are mainly used for water supply schemes and they could be supported on RCC shaft, RCC or steel frame, or masonry pedestal

Dr. Hamida

NATIONAL INFORMATION CENTER OF EARTHQUAKE ENGINEERING

Provisions with Commentaryand Explanatory Examples

الكود المصري واحمال الزلازل على الخزانات

ومطابق للكودات الأوربية

١٠-١-٥ الكودات المرجعية :

١ – الكود الأمريكى لمؤسسات أعمال المياه (AWWAD.100) .

٢ – الكود الأوروبى الموحد – الجزء الثامن.

٣ – الكود الأمريكى لمعهد الخرسانة (ACI 350.3)

٤ – الكود الهندى (IS 1893) .

١٠-٥ معامل تخفيض ردود الأفعال Response Modification Factor (R)

١٠-٥-١ يؤخذ معامل تعديل ردود الأفعال (تخفيض القوى) R طبقاً للجدول (١٠-١)، وحسب درجة المطولية (مطولية محدودة أو مطولية كافية).

جدول (١٠-١) معامل تخفيض رد الفعل (R)

Type of tank نوع الخزان	*R*
Elevated tank خزان مرتفع	
خزان مرتكز على قلب من الحوائط الخرسانية المسلحة مسلح بشبكتين من التسليح Tank Supported on RC shaft	1.8
Tank Supported on RC frame خزان مرتكز على إطار من الخرسانة المسلحة	
(a) Frame conforming to limited ductility (أ) إطار ذو مطولية محدودة	1.8
(b) Frame conforming to sufficient ductility (ب) إطار ذو مطولية كافية	2.5
Tank Supported on steel frame خزان مرتكز على إطار من الصلب	2.5
Ground Supported Tank خزان أرضى	
RC/ Prestressed tank خزان من الخرسانة المسلحة أو الخرسانة سابقة الإجهاد	
a) Fixed or hinged base tank (أ) قاعدة مثبتة أو تسمح بالدوران	2.0
Steel tank خزان من الصلب	
a) Unanchored base (أ) خزان غير مثبت بالأرض	2.0
b) Anchored base (ب) خزان مثبت بالأرض	2.5

١٠-٦ طريقة طيف التجاوب المبسط لتحديد أحمال الزلازل على الخزانات

Simplified Response Spectrum Method for Determining Seismic Loads on Tanks

١٠-٦-١ نموذج للتحليل الزلزالى

يؤدى اهتزاز السوائل داخل الخزانات إلى توليد ضغط هيدروديناميكى دفعى وحركى على كل من حوائط وأرضية الخزان وذلك بالإضافة إلى الضغط الأستاتيكى. ويمكن حساب الضغط

الهيدروديناميكى فى التحليل بتمثيل الخزان بنموذج مكافئ فى الكتلة والجساءة مع الأخذ فى الأعتبار التأثير المتبادل بين الحائط والسائل. وتعتمد العوامل المحددة لهذا النموذج على شكل الخزان ومدى مرونته.

١٠-٦-١-١ الخزانات المرتكزة على الأرض

أ - يمكن تمثيل الخزانات بنموذج مكافئ من الكتلة والجساءة له درجة حرية واحدة كما هو موضح بشكل (١٠-١-أ). تتصل الكتلة الحركية من السائل (m_i) بحائط الخزان عند ارتفاع h_iأو h_i^* بجساءة عالية. وبالمثل تتصل الكتلة الدفعية من السائل (m_c) بحائط الخزان عند إرتفاع h_c أو h_c^* بزنبرك ذو جساءة k_c .

ب - تحدد العوامل البارامترية $m_i, m_c, h_i^*, h_i, h_c^*, h_c, k_c$ من شكل رقم (١٠-٢) بالنسبة للخزانات الدائرية وشكل رقم (١٠-٣) بالنسبة للخزانات المستطيلة.

جـ- الارتفاعات h_i و h_c أخذ فى الأعتبار الضغط الهيدروديناميكى على حوائط الخزان فقط وبالتالى تستخدم فى حساب العزوم أسفل حائط الخزان.

د - الارتفاعات h_i^* و h_c^* تأخذ فى الأعتبار الضغط الهيدروديناميكى على حوائط وقاعدة الخزان وبالتالى تستخدم فى حساب العزوم الإنقلابية على قاعدة الخزان.

١٠-٦-١-٢ الخزانات المرفوعة

أ - يمكن تمثيل الخزانات المرفوعة المثبتة على أبراج بنموذج مكافئ فى الكتلة والجساءة له درجتين من حرية الحركة واحدة تمثل الكتلة الحركية من السائل (m_i) مضافا إليها كتلة المنشأ (m_s) وذات جساءة جانبية تساوى (k_s) والأخرى تمثل الكتلة الدفعية من السائل (m_c) وذات زنبرك بجساءة (k_c) كما هو موضح فى شكل رقم (١٠-١-ب). وتحدد قيمة كتلة المنشأ (m_s) مساوية لكتلة الخزان بالإضافة إلى ثلث كتلة الهيكل الحامل له.

ب - تحدد العوامل البارامترية $k_c, m_i, m_c, h_i, h_i^*, h_c, h_c^*$ من شكل رقم (١٠-٢) بالنسبة للخزانات الدائرية وشكل رقم (١٠-٣) بالنسبة للخزانات المستطيلة.

جـ- الارتفاعات h_c^*, h_i^*, h_c, h_i تحدد كما جاء فى الفقرة (جـ) ، (د) من البند (١٠-٦-١-١)

١٠-٦-١-٣ خزانات ذات أشكال أخرى

بالنسبة لأشكال الخزانات الأخرى غير الدائرية والمستطيلة مثل الشكل المخروطى تؤخذ قيمة h/D المناظرة لخزان دائرى مكافئ له نفس الحجم وقطره مساوياً لقطر الخزان عند المستوى العلوى للسائل كما أن العوامل البارامترية $k_c, m_i, m_c, h_i, h_i^*, h_c, h_c^*$ تكون المناظرة للخزان الدائرة المكافئ.

١٠-٦-٢ معادلات لحساب الطول الموجى الأساسى

١٠-٦-٢-١ زمن الطول الموجى الحركى (Impulsive mode)

أ - بالنسبة للخزانات الدائرية المرتكزة على الأرض والتى يكون فيها اتصال الحوائط ببلاطة قاعدة الخزان يسمح بانتقال العزوم (rigidly connected)، يتم حساب زمن الطول الموجى الحركى (T_i) من المعادلة:

$$T_i = C_i \frac{h\sqrt{\rho}}{\sqrt{t/D}\sqrt{E}} \qquad (10\text{-}1)$$

حيث :

T_i زمن الطول الموجى الحركى بالثانية

C_i معامل يحدد من شكل رقم (١٠-٤)

h أقصى عمق للسائل

t سمك حائط الخزان

E معامل المرونة لحائط الخزان

ρ كثافة كتلة السائل

ب - بالنسبة للخزانات المستطيلة المرتكزة على الأرض والتى يكون فيها اتصال الحوائط ببلاطة قاعدة الخزان يسمح بانتقال العزوم (rigidly connected) ، يتم حساب زمن الطول الموجى الحركى (T_i) فى اتجاهين متعامدين من المعادلة:

$$T_i = 2\pi\sqrt{\frac{d}{g}} \qquad (10\text{-}2)$$

حيث:

d الإزاحة الجانبية المرنة لحائط الخزان عند إرتفــاع $\bar{h}$ نتيجة أخذ ضغط موزع ثابت ذو قيمة q طبقاً للمعادلات الآتية :

$$\bar{h} = \frac{\frac{m_i}{2}h_i + \overline{m_w}\frac{h}{2}}{\frac{m_i}{2} + \overline{m_w}} \qquad (10\text{-}3)$$

$$q = \frac{\left(\frac{m_i}{2} + \overline{m_w}\right)g}{Bh} \qquad (10\text{-}4)$$

m_i الكتلة الحركية من السائل.

$\overline{m_w}$ كتلة أحد حوائط الخزان فى إتجاه عمودى على القوى الزلزالية.

B البعد الداخلى للخزان فى إتجاه القوة الزلزالية.

جـــ- بالنسبة للخزانات المرفوعة سواء الدائرية أو المستطيلة يتم حساب زمن الطول المــوجى الحركى (T_i) من المعادلة:

$$T_i = 2\pi\sqrt{\frac{m_i + m_s}{k_s}} \qquad (10\text{-}5)$$

حيث:

m_s قيمة كتلة الخزان مضافا لها ثلث كتلة الهيكل الحامل له.

k_s الجساءة الجانبية للهيكل الحامل فى إتجاه القوة الزلزالية .

١٠-٦-٢-٢ زمن الطول الموجى الدفعى (Convective mode)

أ - بالنسبة لجميع أنواع الخزانات سواء الدائرية أو المستطيلة الشكل يتم حساب زمن الطول الموجى الدفعى (T_c) من المعادلة:

$$T_c = 2\pi\sqrt{\frac{m_c}{k_c}} \qquad (10\text{-}6)$$

حيث : أن قيم m_c و k_c يمكن تحديدها من الشكل (١٠-٢) للخزان الدائرى وشكل رقم (١٠-٣) للخزان المستطيل.

ب – كحل بديل يمكن حساب زمن الطول الموجى الدفعى (T_c) :

- للخزانات الدائرية من المعادلة :

$$T_c = C_c\sqrt{D/g} \qquad (10\text{-}7)$$

حيث:

C_c : معامل يمكن الحصول عليه من شكل رقم (١٠-٤)

D : القطر الداخلى للخزانات

- للخزانات المستطيلة من المعادلة :

$$T_c = C_c\sqrt{L/g} \qquad (10\text{-}8)$$

حيث:

C_c : معامل يحدد من شكل رقم (١٠-٥).

L : الطول الداخلى للخزان فى إتجاه موازى لأتجاه القوى الزلزالية.

١٠-٦-٢-٣ تأثير الخزانات المرتكزة على تربة ضعيفة

أ – فى الخزانات المرتكزة على تربة ضعيفة يمكن أخذ تأثير مرونة التربة فى حساب زمن الطول الموجى.

ب – بصفة عامة فإن مرونة التربة لاتؤثر على زمن الطول الموجى الدفعى بينما يمكن أن تؤثر على زمن الطول الموجى الحركى.

١٠-٦-٣ قوى القص الأساسية القصوى

أ - بالنسبة للخزانات المرتكزة على الأرض فإن قوى القص أسفل حائط الخزان (V_i) نتيجة التشكل الحركى:

$$V_i = S_d(T_i).(m_i + m_w + m_t) \qquad (10\text{-}9)$$

وقوى القص V_c نتيجة التشكل الدفعى:

$$V_c = S_d(T_c).(m_c) \qquad (10\text{-}10)$$

حيث:

$S_d(T_i)$ قيمة طيف التجاوب التصميمى للتحليل الإنشائى المرن (طبقاً للبند (٨-٤-٢-٥)) عند زمن طول موجى (T_i)

$S_d(T_c)$ قيمة طيف التجاوب التصميمى للتحليل الإنشائى المرن (طبقاً للبند (٨-٤-٢-٥)) عند زمن طول موجى (T_c)

m_i الكتلة الحركية من السائل

m_w كتلة حوائط الخزان

m_t كتلة بلاطة سطح الخزان

g عجلة الجاذبية الأرضية

ب - بالنسبة للخزانات المرفوعة المثبتة على هياكل حاملة فإن قوى القص فوق منسوب ظهـر الأساسات (V_i) نتيجة التشكل الحركى يتم حسابها من المعادلة التالية :

$$V_i = S_d(T_i).(m_i + m_s) \qquad (10\text{-}11)$$

وقوى القص (V_c) نتيجة التشكل الحركى يتم حسابها من المعادلة التالية :

$$V_c = S_d(T_c).(m_c) \qquad (10\text{-}12)$$

حيـــث : m_s كتلة الخزان بالإضافة إلى ثلث كتلة الهيكل الحامل له.

جـــ- قوى القص الكلية V يمكن حسابها بالجذر التربيعى لمجموع مربعات قوى القص نتيجـــة للتشكل الحركى والدفعى معاً:

$$V = \sqrt{(V_i)^2 + (V_c)^2} \qquad (10\text{-}13)$$

د – قيم طيف التجاوب طبقاً لجدول (٨–٣) تم حسابها على أساس نسبة اضــمحلال تــصميمى قيمتها ٥% ويتم تعديله إذا اختلفت نسبة الاضمحلال (على سبيل المثال، يتم ضرب القــيم فى ١,٧٥ فى حالة نسبة اضمحلال تصميمى قيمها ٠,٥%).

١٠–٦–٤ العزم الأساسي الأقصى

أ – بالنسبة للخزانات المرتكزة على الأرض فإن عزوم الإنحناء أسفل حائط الخـــزان نتيجـــة التشكل الحركى (M_i) :

$$M_i = S_d(T_i).(m_i h_i + m_w h_w + m_t h_t) \qquad (10\text{-}14)$$

وعزوم الانحناء نتيجة التشكل الدفعى (M_c) :

$$M_c = S_d(T_c).(m_c h_c) \qquad (10\text{-}15)$$

حيث :

h_w ارتفاع مركز ثقل كتلة حائط الخزان

h_t ارتفاع مركز ثقل كتلة بلاطة سطح الخزان

ب – عزوم الانقلاب للخزانات المرتكزة على الأرض نتيجة التشكل الحركى لحــساب إتــزان الخزان أسفل بلاطة قاعدة الخزان M_i^*:

$$M_i^{*} = S_d(T_i)\left(m_i\left(h_i^{*} + t_b\right) + m_w\left(h_w + t_p\right) + m_t\left(h_t + t_b\right) + \frac{m_b t_b}{2}\right) \quad (10\text{-}16)$$

وعزوم الانقلاب نتيجة التشكل الدفعى M_c^{*} :

$$M_c^{*} = S_d(T_c).m_c.\left(h_c^{*} + t_b\right) \quad (10\text{-}17)$$

حيث :

m_b كتلة بلاطة قاعدة الخزان

t_b سمك بلاطة قاعدة الخزان

جـ- بالنسبة للخزانات المرفوعة المثبتة على هياكل فإن عزوم الإنقلاب فوق منسوب ظهر الأساسات (M_i^{*}) نتيجة التشكل الحركى :

$$M_i^{*} = S_d\ (T_i)\ \{m_i\ (h_i^{*} + h_s) + m_S\, h_{cg}\ \} \quad (10\text{-}18)$$

وعزوم الانقلاب (M_c^{*}) نتيجة التشكل الدفعى :

$$M_c^{*} = S_d\ (T_c)\ .\ m_c\ (h_c^{*} + h_s) \quad (10\text{-}19)$$

حيث :

h_s ارتفاع الهيكل الحامل للخزان مقاساً من فوق منسوب ظهر الأساسات إلى أسفل حوائط الخزان.

h_{cg} ارتفاع مركز ثقل الخزان فارغاً مقاساً من فوق منسوب ظهر الأساسات للهيكل الحامل.

د - قوى العزوم الكلية M يمكن حسابها بالجذر التربيعى لمجموع مربعات قوى العزم نتيجة التشكل الحركى والدفعى معاً :

$$M = \sqrt{M_i^{2} + M_c^{2}} \quad (10\text{-}20\ a)$$

or

$$M^{*} = \sqrt{(M_i^{*})^2 + (M_c^{*})^2} \quad (10\text{-}20\ b)$$

١٠-٧ تجميع مركبات الأحمال الناتجة من الزلازل
Combination of the components of the Seismic Action

١٠-٧-١ بعد عمل تحليل زلزالى فى إتجاهين أفقيين متعامدين وفى الأتجاة الرأسى طبقاً للبند (١٠-٦)، يتم استخدام إحدى الطريقتين التاليتين لحساب القوى الداخلية بالأعضاء الإنشائية للخزان.

١٠-٧-١-١ عن طريق التجميع المطلق (بدون إشارات) = (١٠٠%) من قيم القوى الناتجة عن الحركة الزلزالية فى أحد الإتجاهات الثلاثة مع (٣٠%) من قيم القوى الناتجة عن الحركة الزلزالية فى الأتجاهين الآخرين.

أ - $E_T = E_{(Fx)} + 0.30\,E_{(Fy)} + 0.30\,E_{(Fz)}$

ب - $E_T = 0.30\,E_{(Fx)} + E_{(Fy)} + 0.30\,E_{(Fz)}$

جـ - $E_T = 0.30\,E_{(Fx)} + 0.30\,E_{(Fy)} + E_{(Fz)}$

حيث :

$E_{(Fx)}$ القوى الداخلية المتولدة بالعنصر الإنشائى من القوى الزلزالية فى إتجاه محور X

$E_{(Fy)}$ القوى الداخلية المتولدة بالعنصر الإنشائى من القوى الزلزالية فى إتجاه محور Y

$E_{(Fz)}$ القوى الداخلية المتولدة بالعنصر الإنشائى من القوى الزلزالية فى إتجاه محور Z

E_T تأثير أحمال الزلازل المأخوذة فى الأعتبار

مع مراعاة أن اشارة كل مركبة يجب أن تؤخذ فى الإتجاه الأكبر تأثيراً على الخزان.

١٠-٧-١-٢ يأخذ الجذر التربيعى لمجموع مربعات القوى الداخلية بأى عنصر نتيجة تأثير الزلازل فى الثلاثة إتجاهات المتعامدة كل على حده، أى

$$E_T = \sqrt{E_{(Fx)}^2 + E_{(Fy)}^2 + E_{(Fz)}^2} \qquad (10\text{-}21)$$

١٠-٧-٢ يتم تجميع أحمال الزلازل فى الإتجاهات المختلفة والسابق حسابها فى البند (١٠-٧-١) مع الأحمال الدائمة والأحمال الحية طبقاً لكودات التصميم المعنية.

١٠-٨ الضغط الهيدروديناميكى على أرضيات وحوائط الخزان
Hydrodynamic Pressure on Tank Floor and Walls

١٠-٨-١ توزيع الضغط الهيدروديناميكى نتيجة أحمال الزلازل الأفقية

يمكن حساب توزيع الضغط الهيدروديناميكى الدفعى والحركى للسوائل علـــى أرضـــيات وحوائط الخزان نتيجة أحمال الزلازل الأفقية طبقاً لما هو مبين فى البنود التالية :-

١٠-٨-١-١ الضغط الهيدروديناميكى الحركى

أ - الخزانات الدائرية

- معادلة توزيع الضغط الهيدروديناميكى الحركى الجانبى على حوائط الخزان (P_{iw}):

$$P_{iw} = Q_{iw}(y).S_d(T_i).\rho h \cos \phi \qquad (10\text{-}22)$$

حيث :

ρ كثافة كتلة السائل

ϕ زاوية دائرية

y الارتفاع الرأسى لنقطة على حافة الخزان مقاساً من سطح بلاطـــة أرضـــية الخزان

$Q_{iw}(y)$ معامل يحدد من شكل (١٠-٦) ، (١٠-٧)

- ومعادلة توزيع الضغط الهيدروديناميكى الحركى الرأسى على بلاطة أرضية الخزان : (P_{ib})

$$P_{ib} = 0.866 S_d(T_i)\rho h \frac{\sinh\left(0.866\frac{x}{h}\right)}{\cosh\left(0.866\frac{L'}{h}\right)} \qquad (10\text{-}23)$$

حيث :

x المسافة الأفقية لنقطة على أرضية الخزان منعكسة من مركز الخــزان فــى إتجاه القوى الزلزالية.

ب - الخزانات المستطيلة

- معادلة توزيع الضغط الهيدروديناميكى الحركى الجانبى على حوائط الخزان (P_{iw}):

$$P_{iw} = Q_{iw}(y).S_d(T_i).\rho.h \qquad (10\text{-}24)$$

حيث :

$Q_{iw}(y)$ معامل يحدد من شكل (١٠-٧)

- معادلة توزيع الضغط الهيدروديناميكى الحركى **الرأسى** على بلاطة أرضية الخــزان (P_{ib})

$$P_{ib} = Q_{ib}(x).S_d(T_i).\rho.h \qquad (10\text{-}25)$$

حيث :

$Q_{ib}(x)$ معامل يحدد من شكل (١٠-٧)

١٠-٨-١-٢ الضغط الهيدروديناميكى الدفعى

أ - الخزانات الدائرية :

- معادلة توزيع الضغط الهيدروديناميكى الدفعى الجانبى على حوائط الخزان (P_{cw}):

$$P_{cw} = Q_{cw}(y).S_d(T_c).\rho.D\left(1-\frac{1}{3}\cos^2\phi\right)\cos\phi \qquad (10\text{-}26)$$

حيث :

$Q_{cw}(y)$ معامل يحدد من الشكل (١٠-٦) ، (١٠-٨)

- معادلة توزيع الضغط الهيدروديناميكى الدفعى الرأسى على بلاطة أرضية الخزان (P_{cb}):

$$P_{cb} = Q_{cb}(x).S_d(T_c).\rho.D \qquad (10\text{-}27)$$

حيث :

$Q_{cb}(x)$ معامل يحدد من الشكل (١٠-٨).

ب - الخزانات المستطيلة :

- معادلة توزيع الضغط الهيدروديناميكى الحركى الجانبى على حوائط الخزان (P_{cw}):

$$P_{cw} = Q_{cw}(y).S_d(T_c).\rho.L \qquad (10\text{-}28)$$

حيث :

$Q_{cw}(y)$: معامل يحدد من شكل رقم (١٠-٩)

- معادلة توزيع الضغط الهيدروديناميكى الحركى الرأسى على أرضية الخزان (P_{cb}):

$$P_{cb} = Q_{cb}(x).S_d(T_c).\rho.L \qquad (10\text{-}29)$$

حيث :

$Q_{cb}(x)$ معامل يحدد من شكل رقم (١٠-٩).

١٠-٨-١-٣ فى الخزانات الدائرية يكون توزيع الضغط الهيدروديناميكى الجانبى على الحوائط متغير القيمة على محيط الخزان . ولكن يمكن فرض توزيع تقريبى مبسط ذو توزيع ضغط هيدروديناميكى ثابت القيمة مساوى لأقصى قيمة ضغط كما هو موضح بشكل (١٠-١٠).

١٠-٨-١-٤ فى الخزانات الدائرية والمستطيلة يكون توزيع الضغط الهيدروديناميكى الجانبى على الحوائط منحنى متغير القيمة مع إرتفاع الخزان . ولكن يمكن فرض توزيع خطى متغير القيمة ويعطى قوى قص وعزوم إنقلاب أسفل حائط الخزان مكافئة للتوزيع الحقيقى كما هو موضح بالشكل (١٠-١٠).

١٠-٨-١-٥ قيمة الضغط على حوائط الخزان نتيجة قصورها الذاتى :

$$P_{ww} = S_d(T_i).t.\rho_m \qquad (10\text{-}30)$$

حيث :

ρ_m كثافة كتلة حوائط الخزان .

t سمك حائط الخزان .

١٠-٨-٢ توزيع الضغط الهيدروديناميكى نتيجة أحمال الزلازل الرأسية

يمكن حساب توزيع الضغط الهيدروديناميكى الإضافى الجانبى للسوائل (P_v) على حوائط الخزان نتيجة أحمال الزلازل الرأسية طبقاً لما يلى :

$$P_v = S_v(T).\rho.h\left(1-\frac{y}{h}\right)/R \qquad (10\text{-}31)$$

حيث :

y مسافة رأسية مقاسة من أسفل نقطة لحائط الخزان.

$S_v(T)$ إحداثي الطيف التصميمى الرأسى للتحليل الأنشائى المرن طبقاً للبند (٨-٤-٢-٣) عند زمن طول موجى فى الإتجاه الرأسى T_v وفى حالة عدم وجود حسابات دقيقة تؤخذ قيمة T_v مساوية ٠,٣ ثابتة لجميع أنواع الخزانات.

١٠-٨-٣ حالات تجميع أقصى قيمة للضغط الهيدروديناميكى نتيجة أحمال الزلازل الأفقية والرأسية

يؤخذ الجذر التربيعى لمجموع مربعات قيمة الضغط الهيدروديناميكى (SRSS) نتيجة أحمال الزلازل الأفقية والرأسية طبقاً لما يلى :

$$P_{max} = \sqrt{(P_{iw} + P_{ww})^2 + P_{cw}^2 + P_v^2} \qquad (10\text{-}32)$$

١٠-٨-٤ ارتفاع موجة التشكل الدفعى

Sloshing Wave height

تؤخذ قيمة أقصى إرتفاع لموجة التشكل الدفعى للسائل فى الخزانات الدائرية

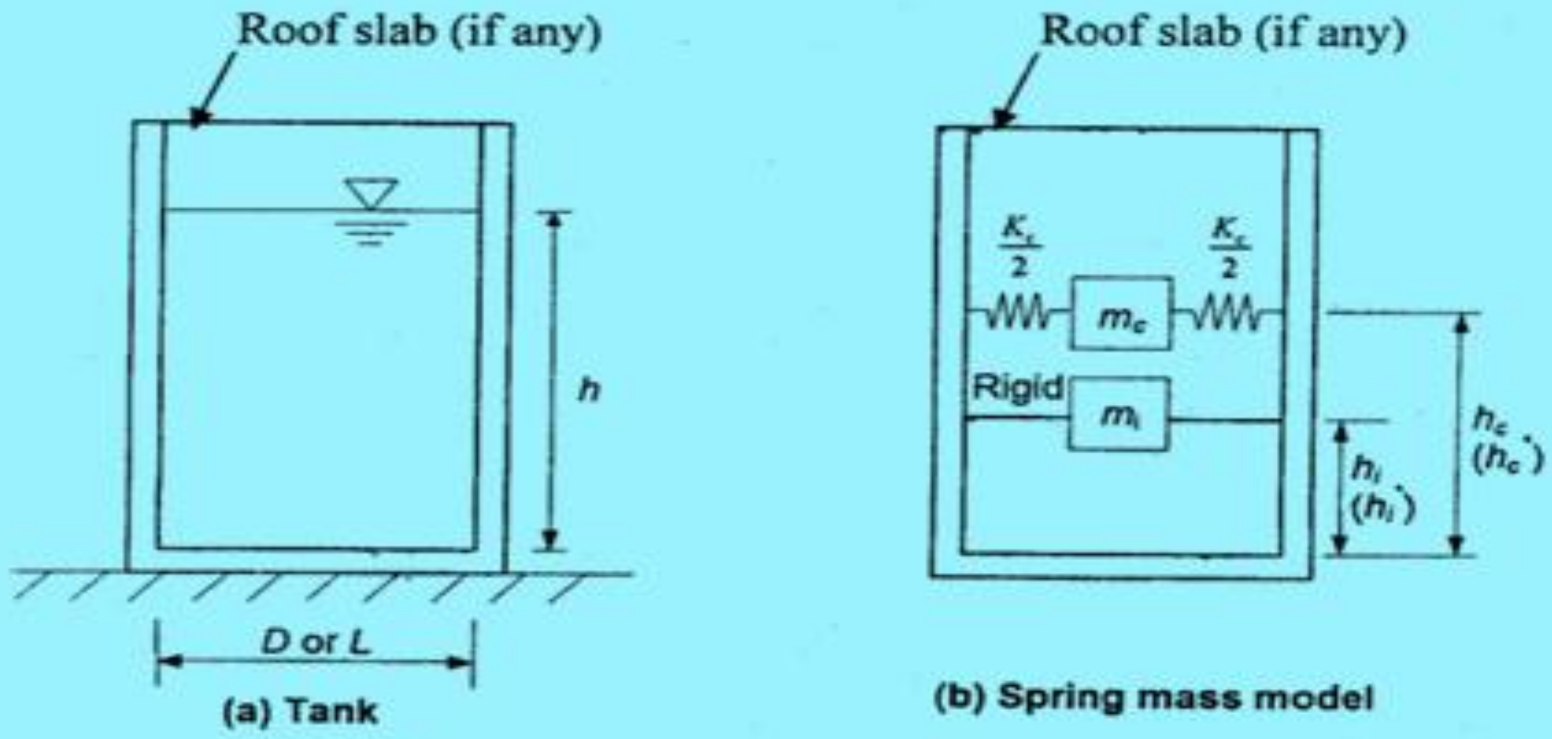
$$d_{max} = S_d(T_c).R.\frac{D}{2} \qquad (10\text{-}33)$$

وفى الخزانات المستطيلة

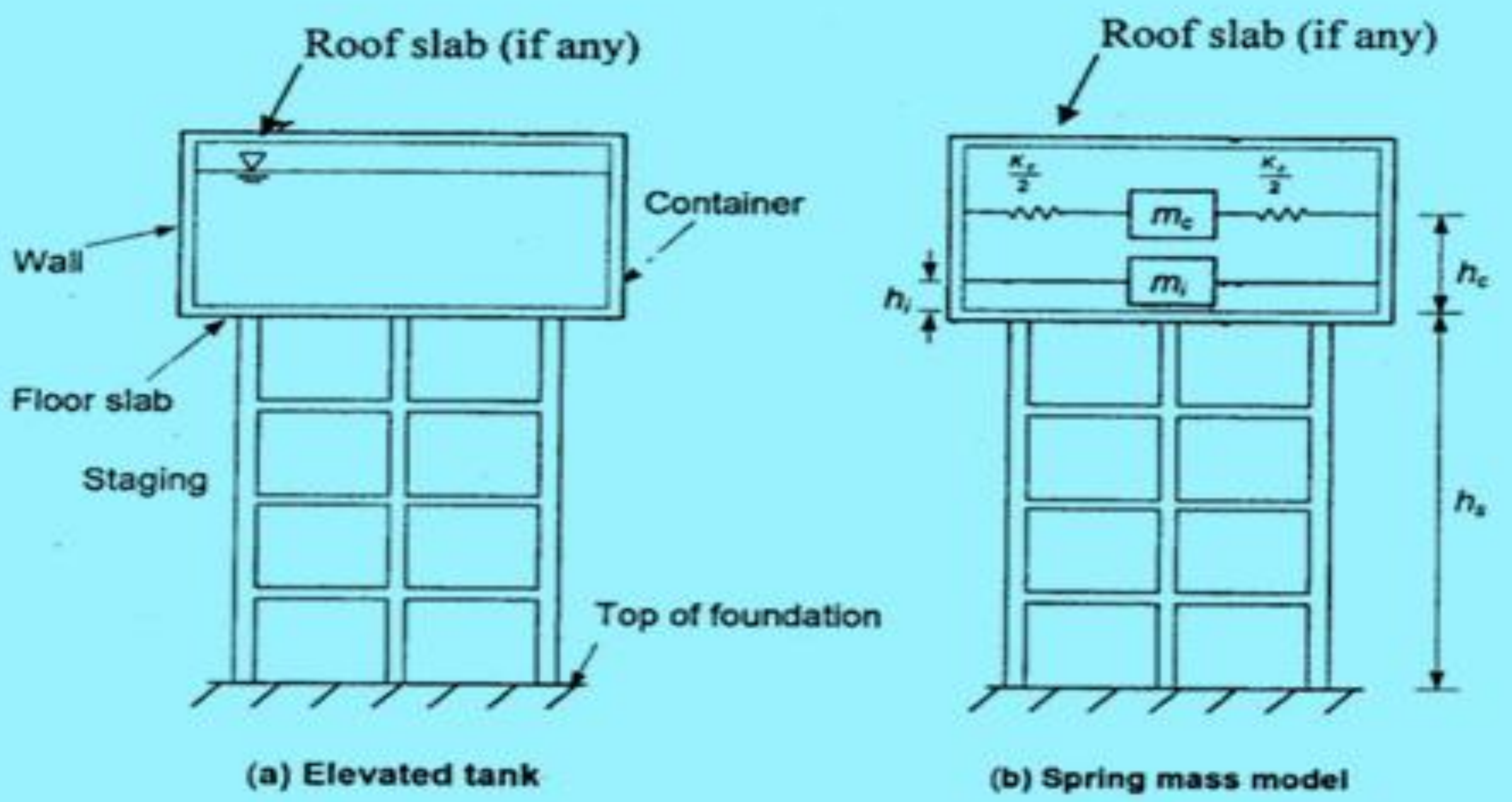
$$d_{max} = S_d(T_c).R.\frac{L}{2} \qquad (10\text{-}34)$$

حيث :

$S_d(T_c)$ قيمة طيف التجاوب التصميمى للتحليل الإنشائى المرن عند زمن الطول الموجى للتشكل الدفعى (T_c).

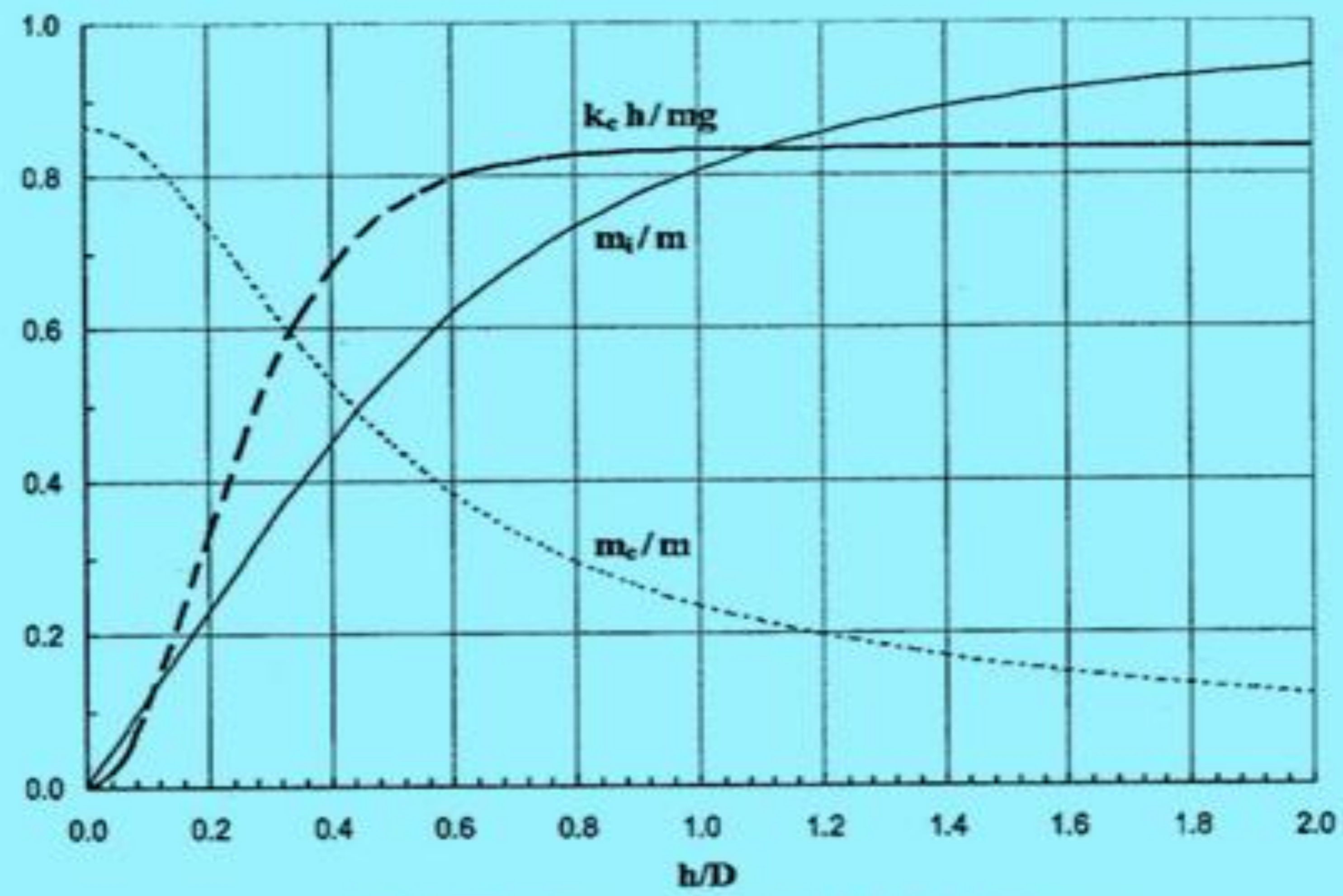


أ – الخزانات المرتكزة على الأرض

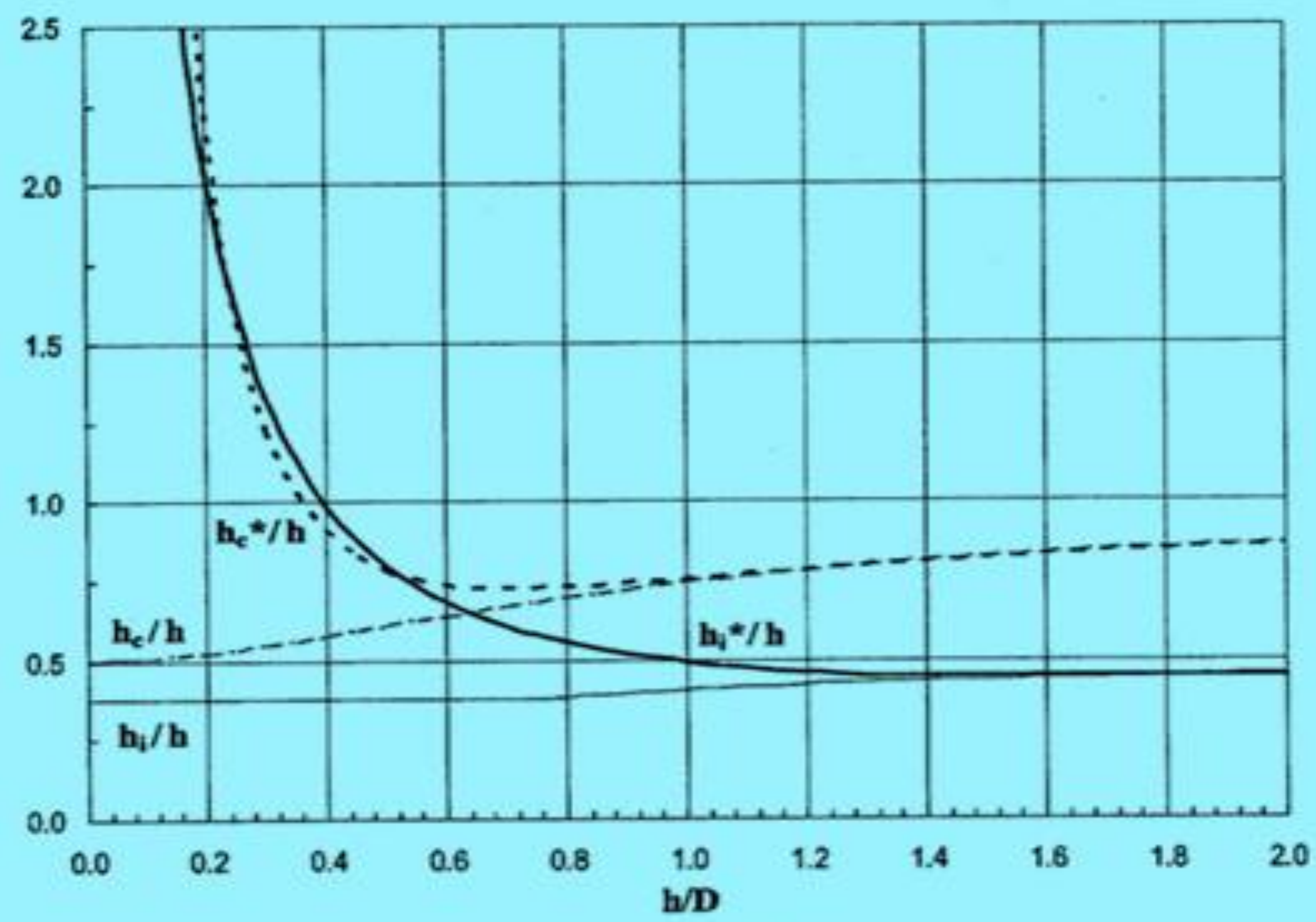


ب – الخزانات المرفوعة عن الأرض

شكل (١٠–١) نماذج الزنبرك – الكتلة للخزانات الإسطوانية والمستطيلة

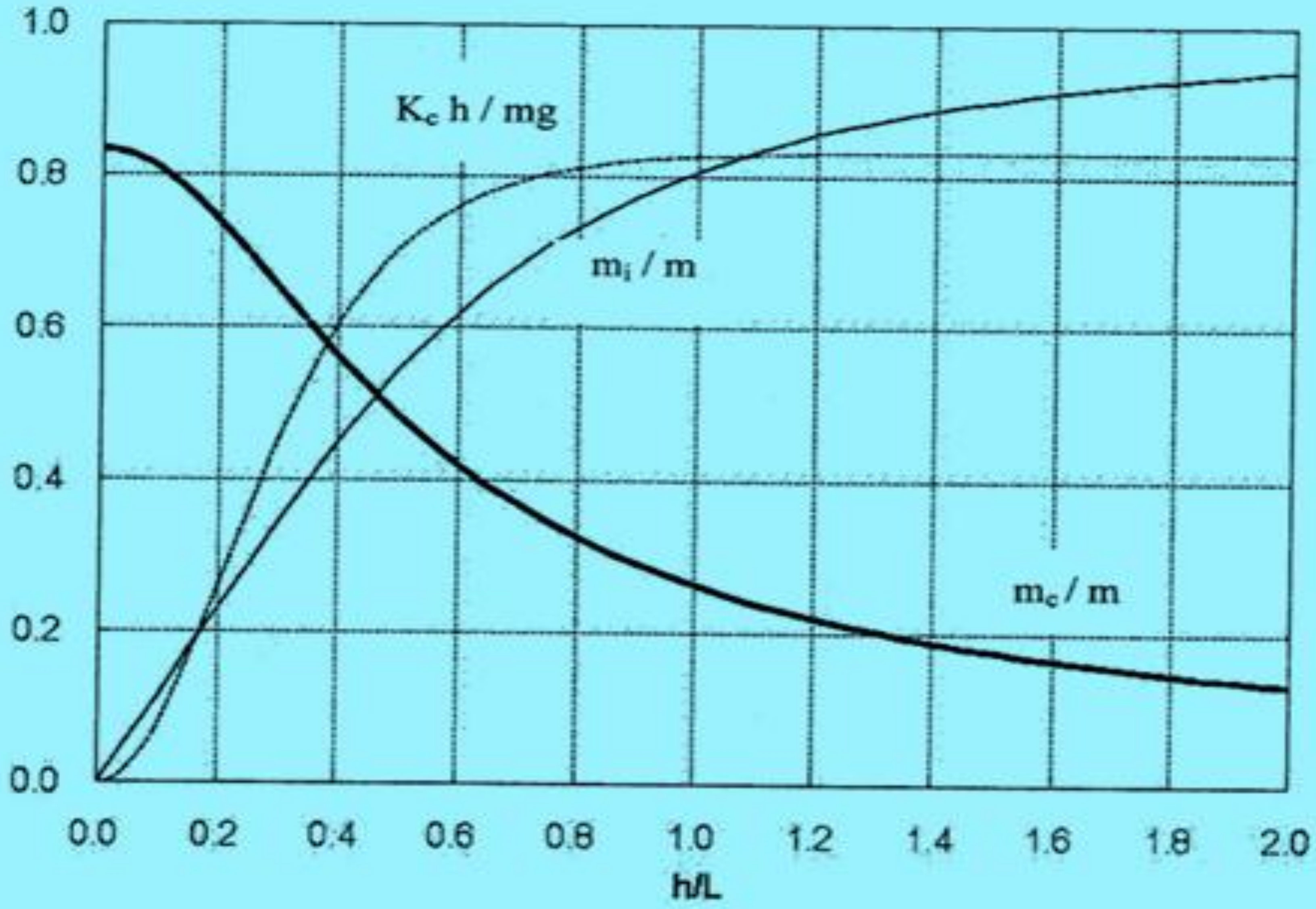


نسب الكتلة الدفعية والحركية وجساءة الزنبرك الحركى

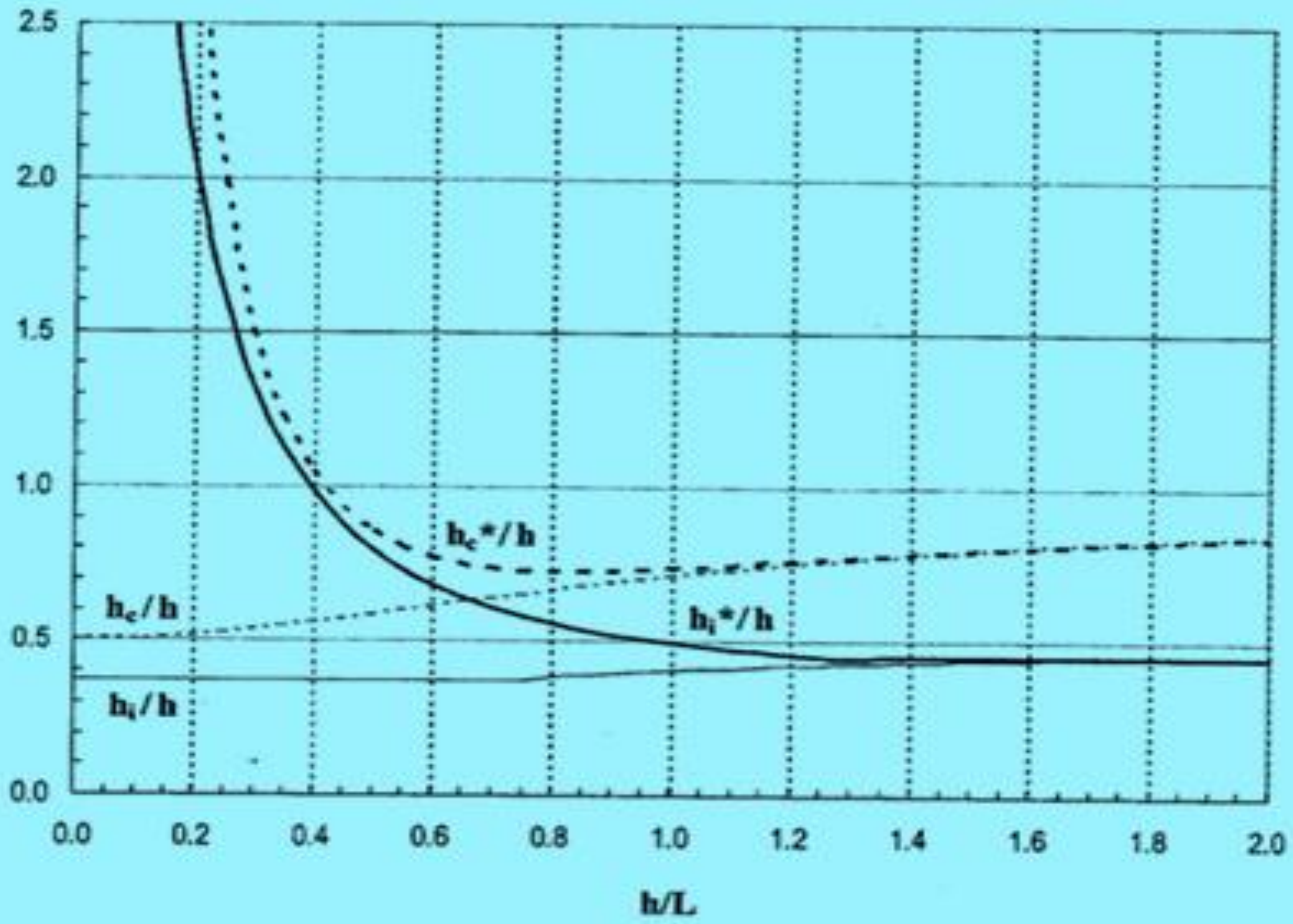


ارتفاعات تأثير الكتلة الدفعية والحركية

شكل (١٠-٢) معاملات نموذج الزنبرك – الكتلة للخزانات الاسطوانية

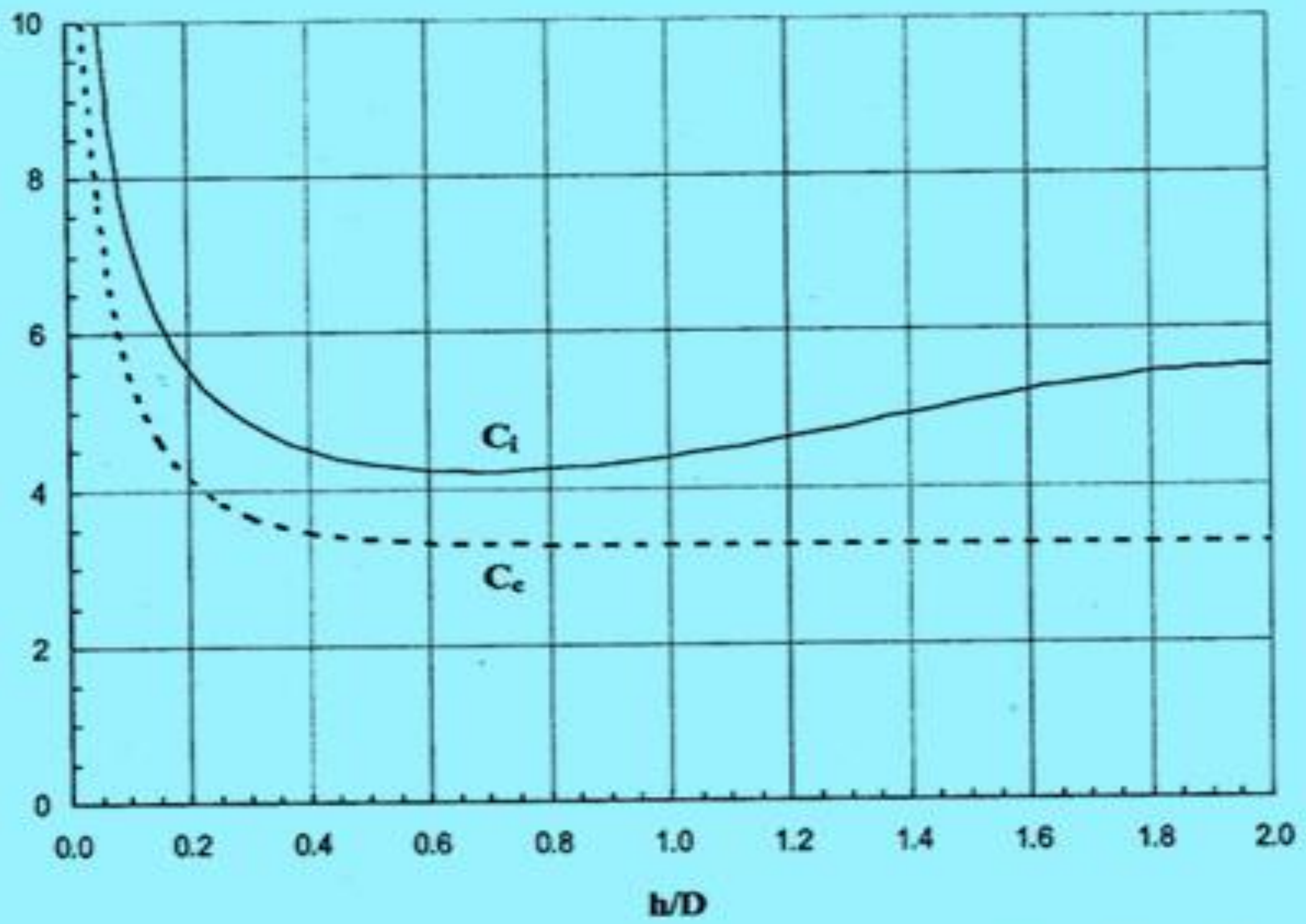


نسب الكتلة الدفعية والحركية وجساءة الزنبرك الحركى

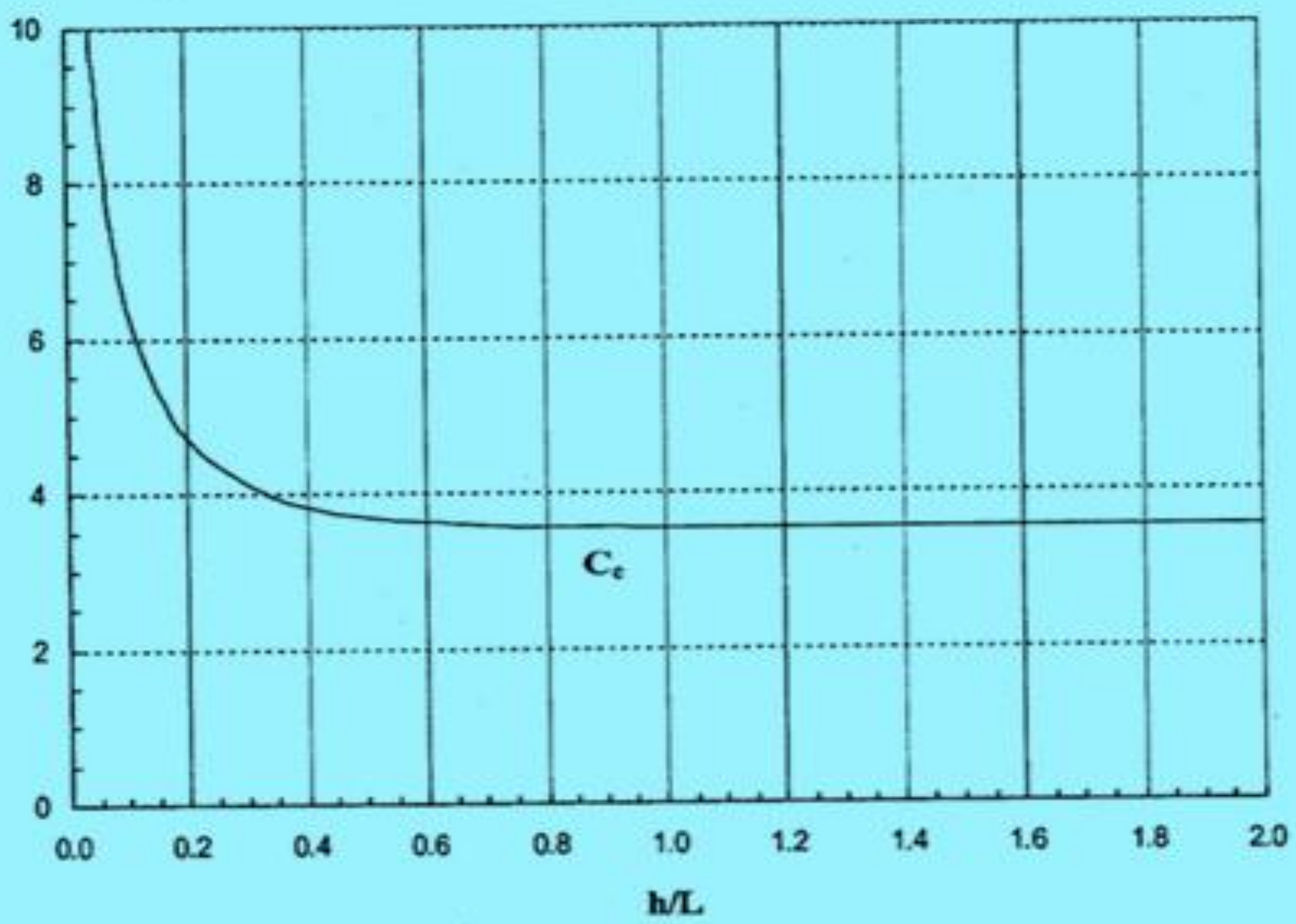


ارتفاعات تأثير الكتلة الدفعية والحركية

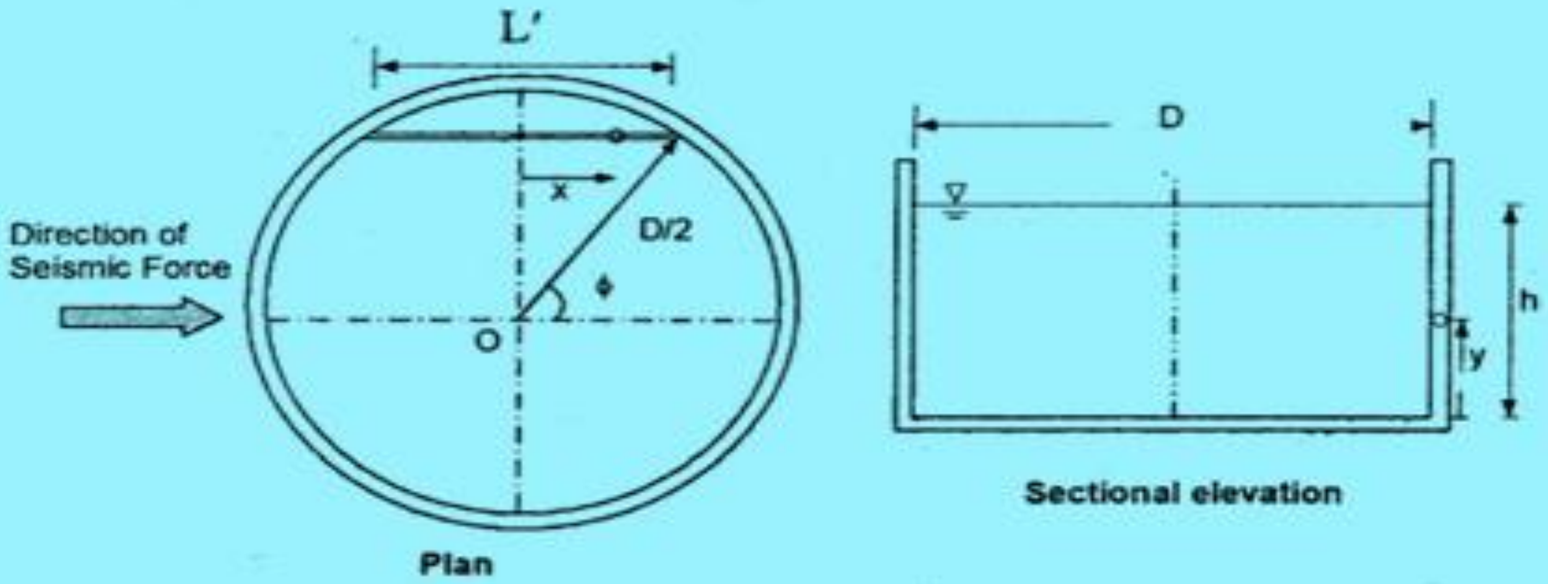
شكل (١٠-٣) معاملات نموذج الزنبرك – الكتلة للخزانات المستطيلة.



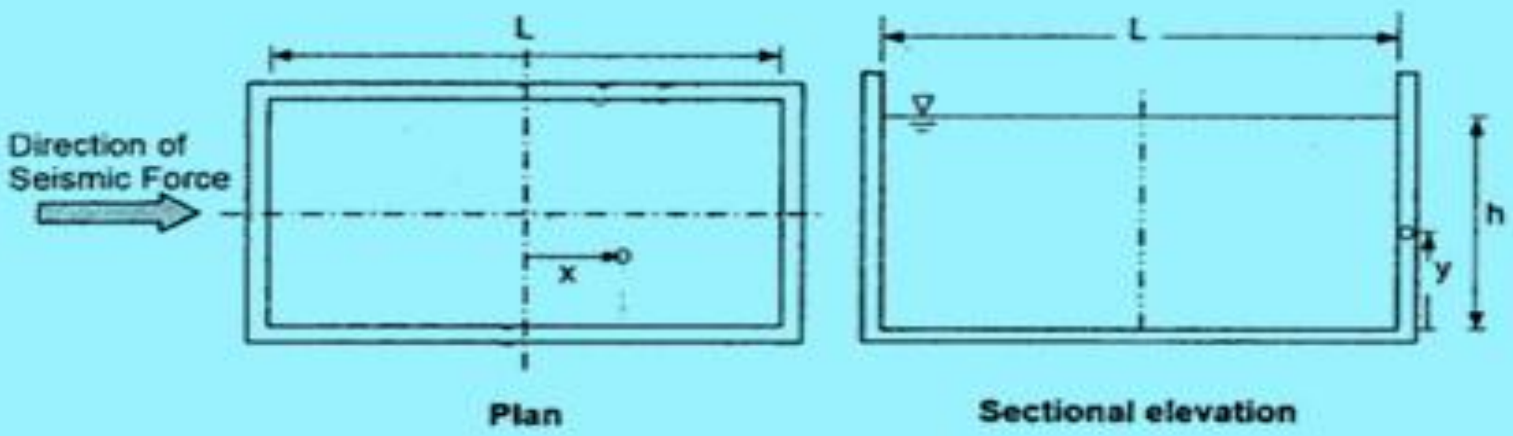
شكل (١٠-٤) معاملات الزمن المودى الدفعية (C_i) والحركية (C_c) للخزانات الإسطوانية.



شكل (١٠-٥) معاملات الزمن المودى الدفعية (C_c) للخزانات المستطيلة.

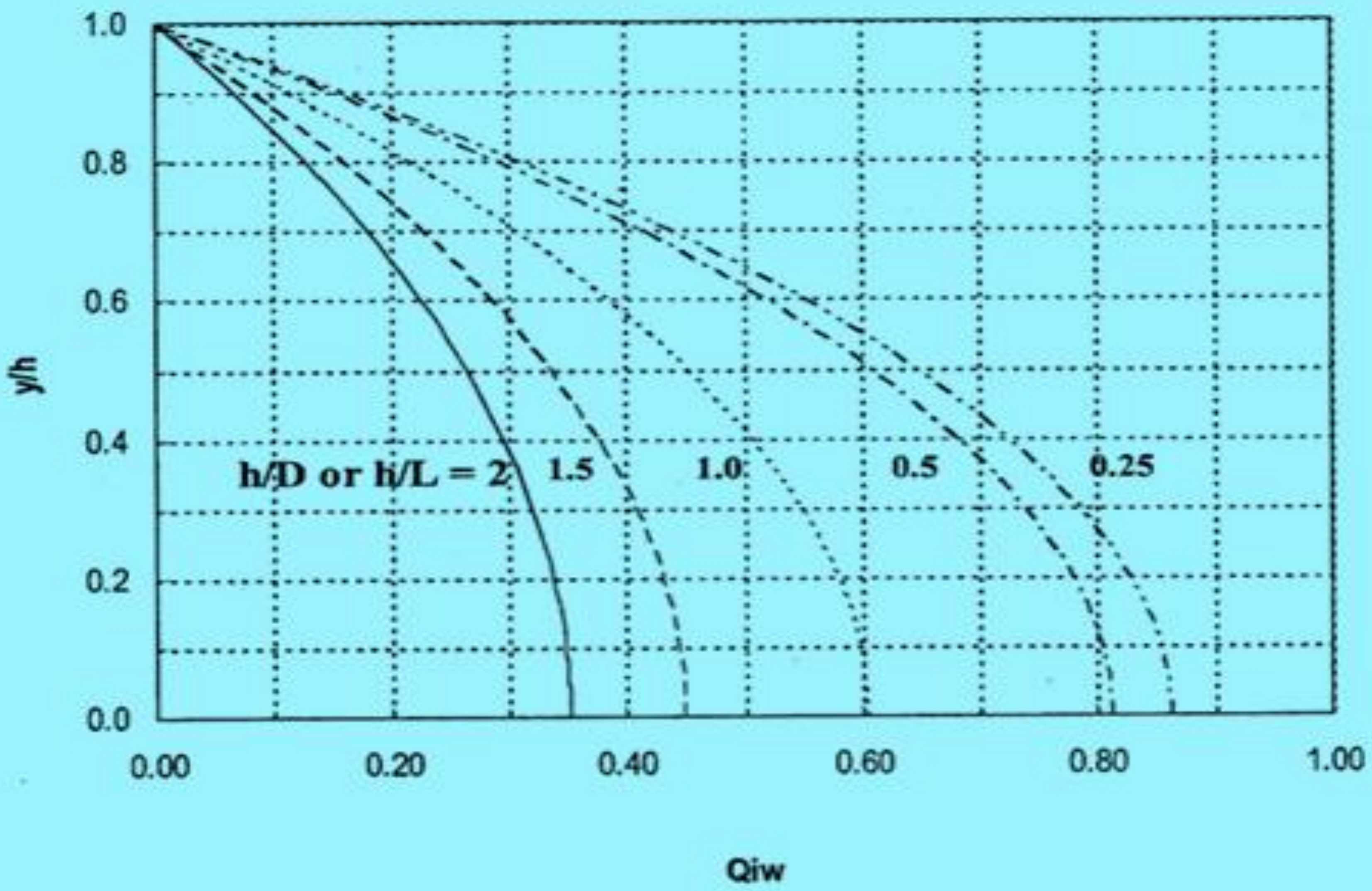


أ – خزان إسطوانى

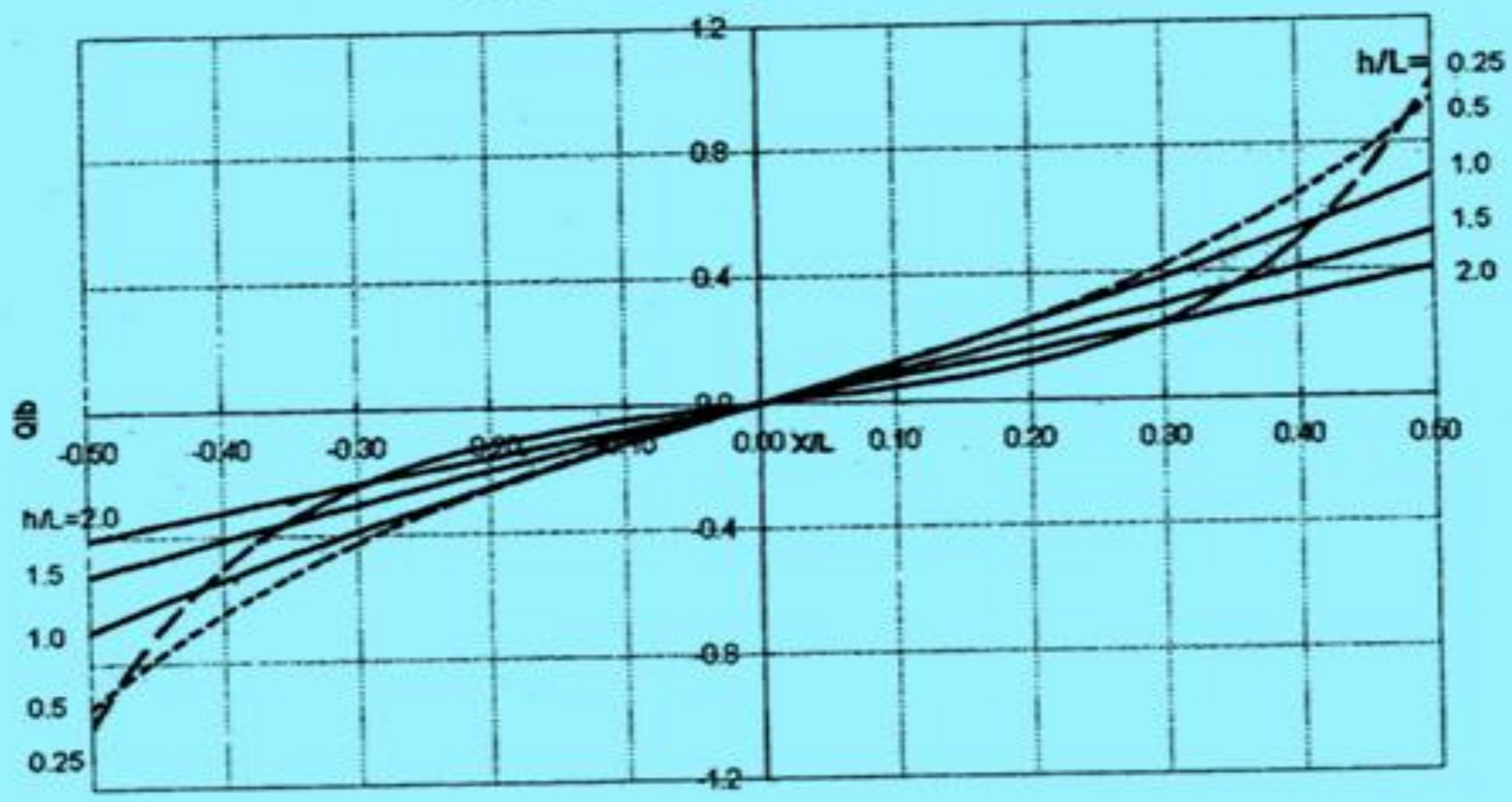


أ – خزان مستطيل

شكل (١٠-٦) الأبعاد الهندسية للخزانات الإسطوانية والمستطيلة.

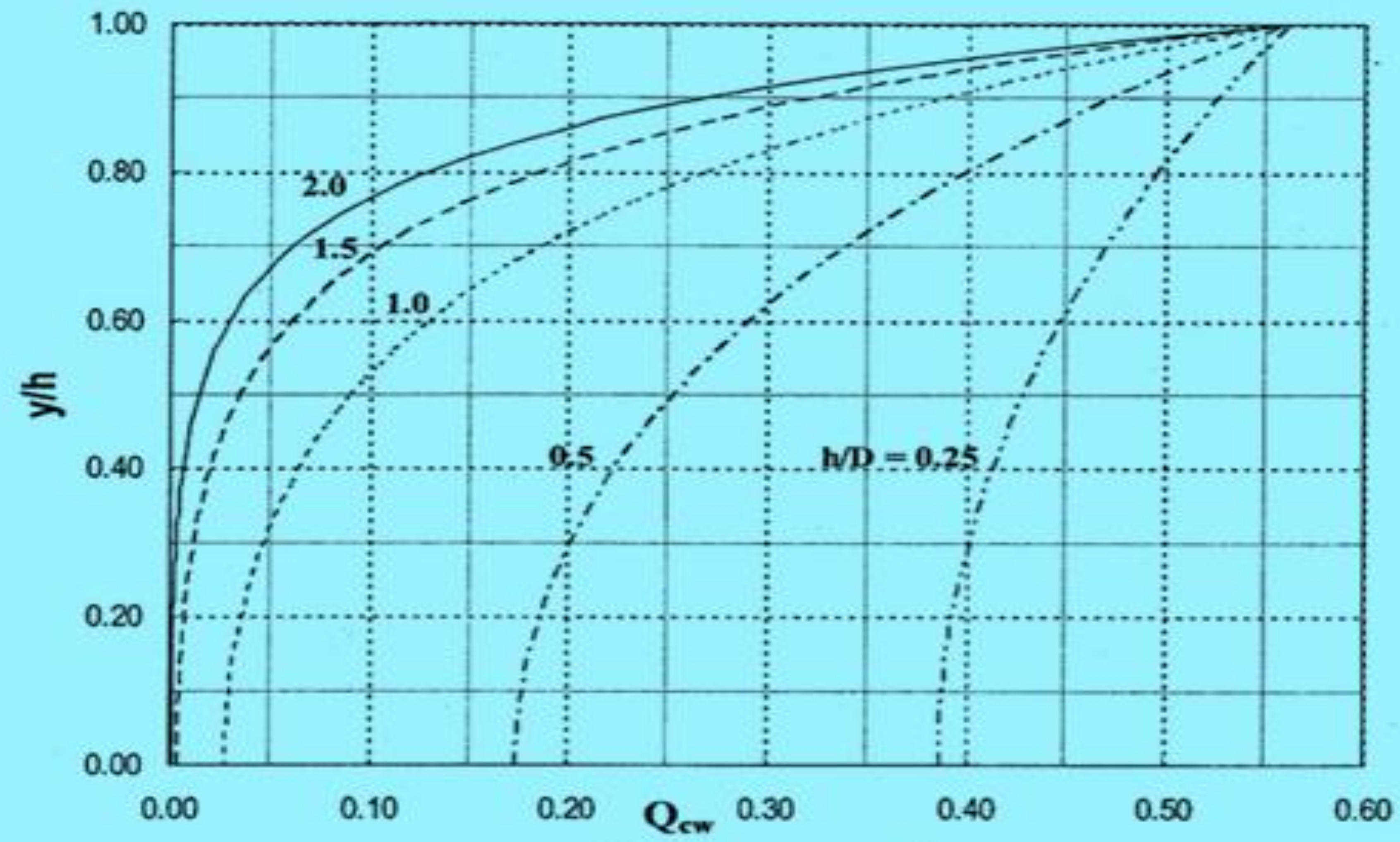


أ – على الحوائط (Q_{iw})

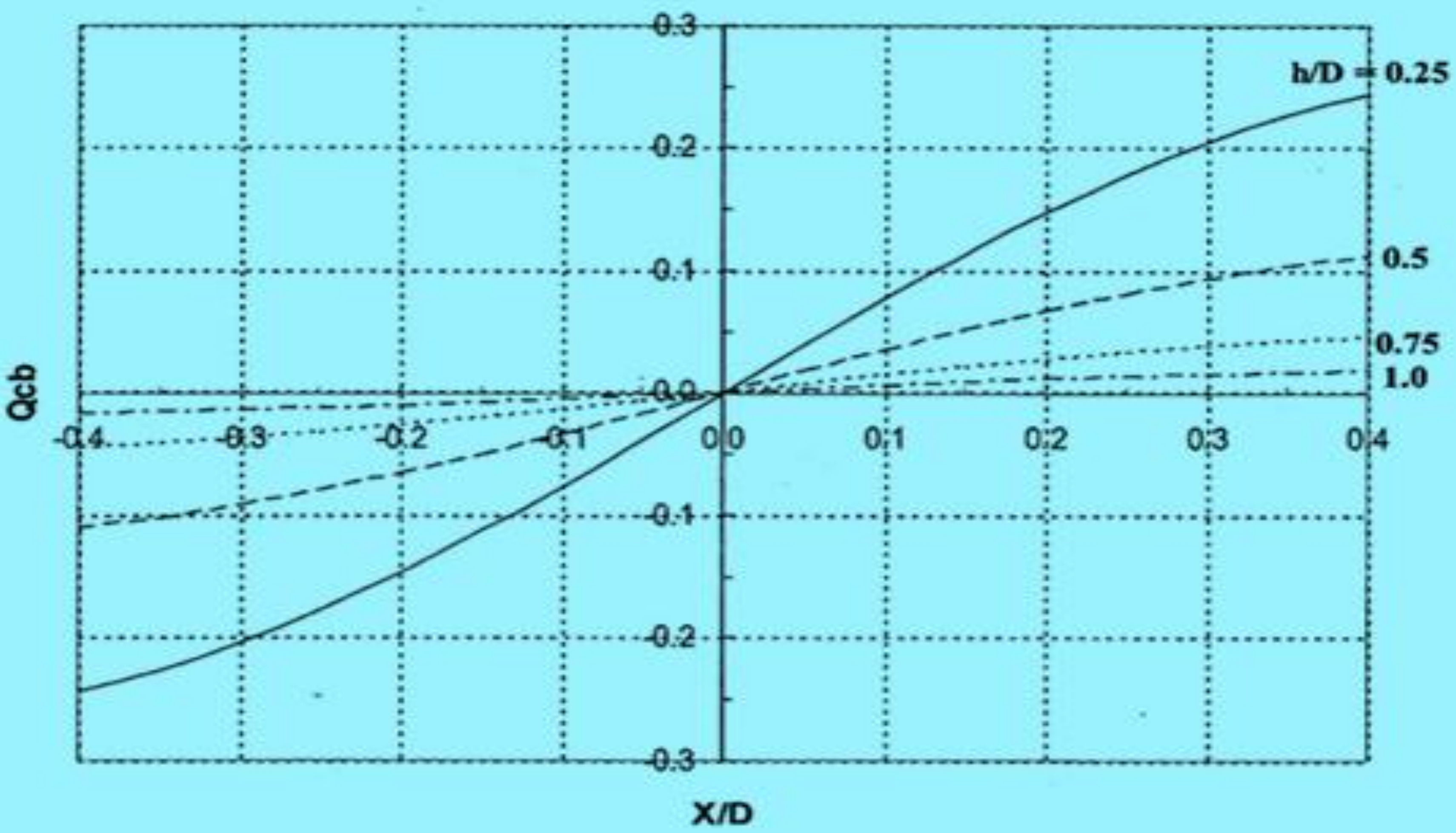


ب – على أرضية الخزان (Q_{ib})

شكل (١٠–٧) معاملات الضغط الدفعى للخزانات الاسطوانية والمستطيلة

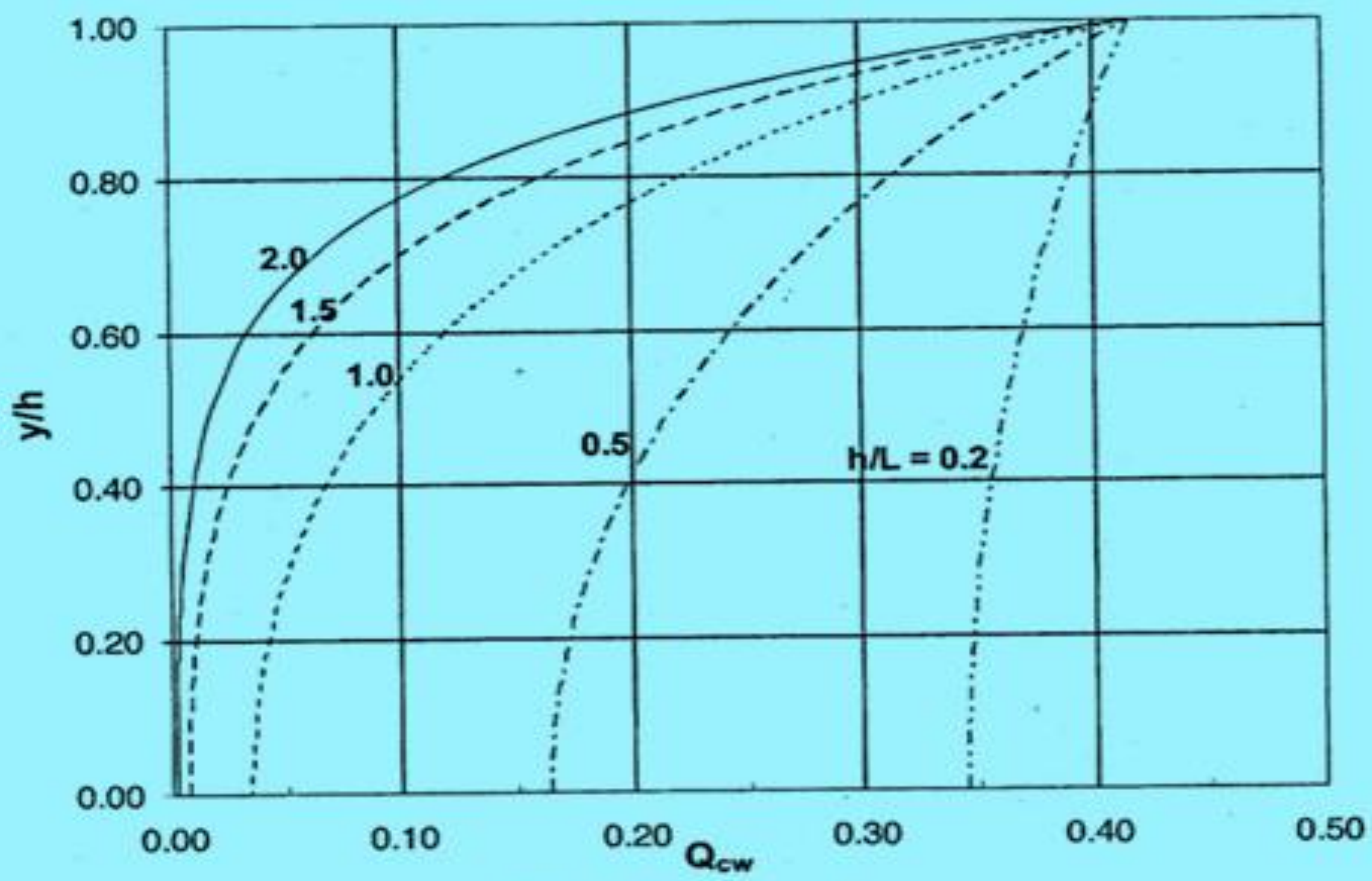


أ – على الحائط(Q_{cw})

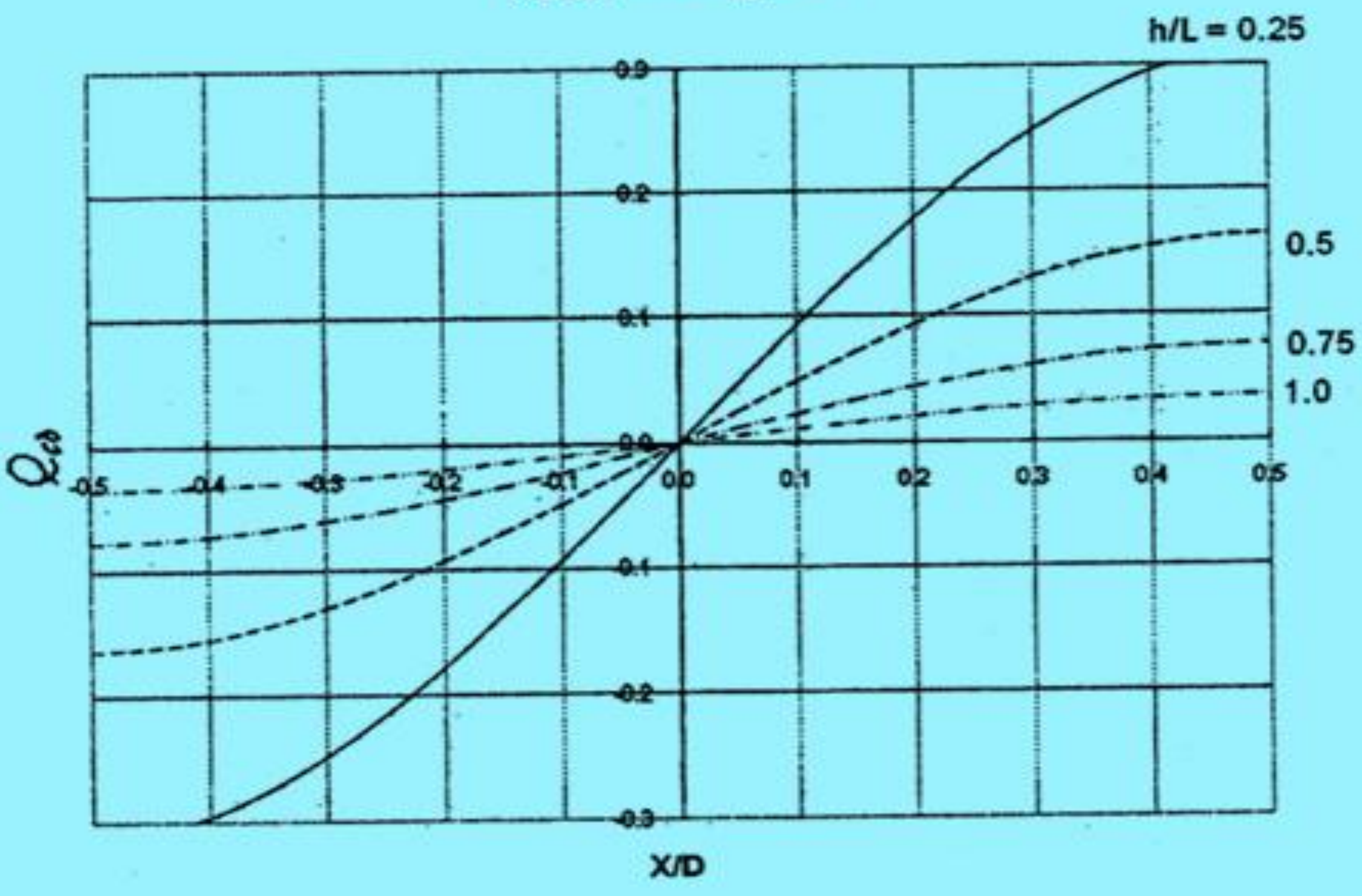


ب – على أرضية الخزان (Q_{cb})

شكل (١٠–٨) معامل الضغط الحركى للخزانات الإسطوانية

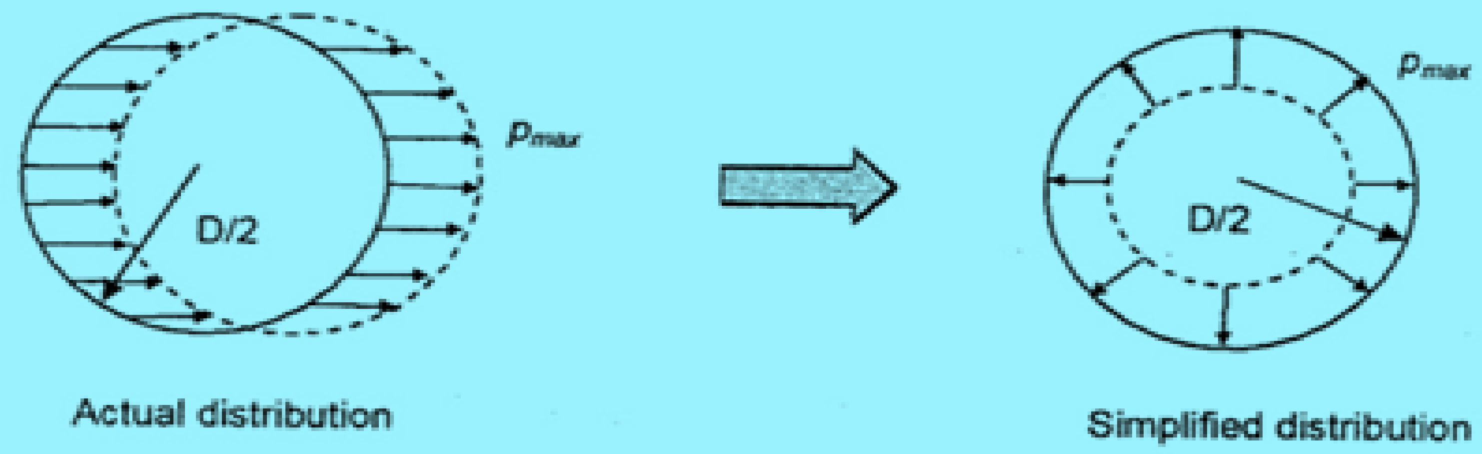


أ – على الحائط (Q_{cw})

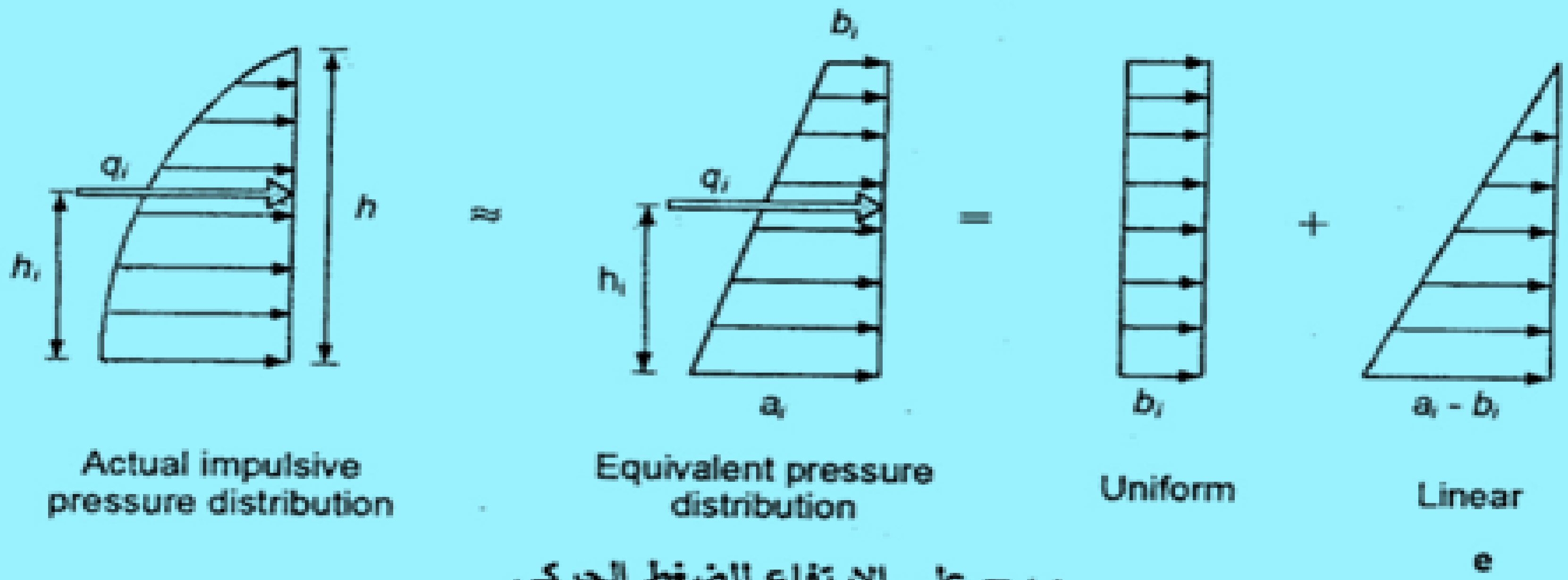


ب – على أرضية الخزان (Q_{cb})

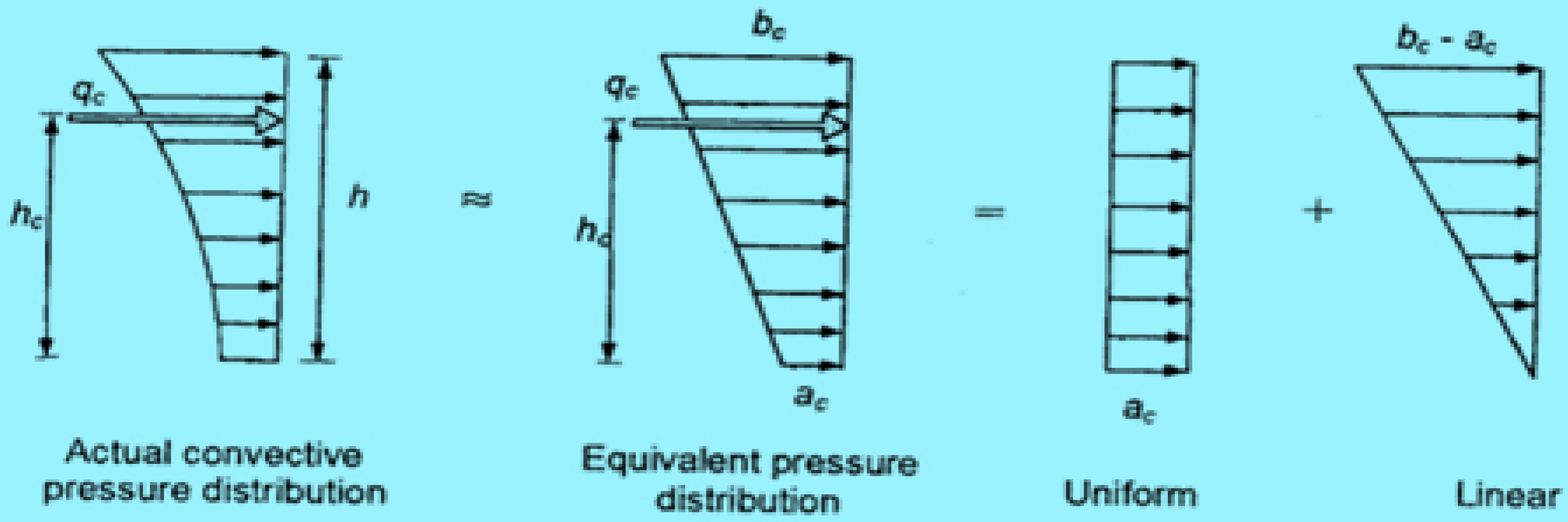
شكل (١٠–٩) معاملات الضغط الحركى للخزانات المستطيلة



أ – على المحيط



ب – على الإرتفاع للضغط الحركى



ب – على الإرتفاع للضغط الدفعى

شكل (١٠-١٠) توزيع الضغط الهيدروديناميكى علىَ الحوائط

design tank to resist seismic

According to the European codes

CONTENTS

PART 2: Explanatory Examples for

Seismic Design of Liquid Storage Tanks

PROVISIONS		COMMENTARY
	liquid (m_l / 2) and mass of one wall ($\overline{m}_w$)	
h_c	Height of convective mass above bottom of tank wall (without considering base pressure)	h_c, h_i, h_c^*, h_i^* are described in Figure C-1a to 1d
h_i	Height of impulsive mass above bottom of tank wall (without considering base pressure)	
h_s	Structural height of staging, measured from top of foundation to the bottom of container wall	
h_t	Height of center of gravity of roof mass above bottom of tank wall	
h_w	Height of center of gravity of wall mass above bottom of tank wall	
h_c^*	Height of convective mass above bottom of tank wall (considering base pressure)	
h_i^*	Height of impulsive mass above bottom of tank wall (considering base pressure)	
h_{cg}	Height of center of gravity of the empty container of elevated tank, measured from base of staging	
I	Importance factor given in Table 1 of this code	
		I_w Moment of inertia of a strip of unit width of rectangular tank wall for out of plane bending; Refer Clause 4.3.1.2
K_c	Spring stiffness of convective mode	
		k_h Dynamic coefficient of earth pressure
K_s	Lateral stiffness of elevated tank staging	
l'	Length of a strip at the base of circular tank, along the direction of seismic force	Refer Figure 8a
L	Inside length of rectangular tank parallel to the direction of seismic force	Refer Figure C-3
m	Total mass of liquid in tank	
m_b	Mass of base slab / plate	In SI unit, mass is to be specified in kg, while the weight is in Newton (N). Weight (W) is equal to mass (m) times acceleration due to gravity (g). This implies that a weight of 9.81 N has a mass of 1 kg.
m_c	Convective mass of liquid	
m_i	Impulsive mass of liquid	

PROVISIONS		COMMENTARY
m_s	Mass of empty container of elevated tank and one-third mass of staging	Refer Clause 4.2.2.3
m_t	Mass of roof slab	
m_w	Mass of tank wall	
$\overline{m}_w$	Mass of one wall of rectangular tank perpendicular to the direction of loading	Refer Clause 4.3.1.2
M	Total bending moment at the bottom of tank wall	
M^*	Total overturning moment at base	
M_c	Bending moment in convective mode at the bottom of tank wall	
M_c^*	Overturning moment in convective mode at the base	
M_i	Bending moment in impulsive mode at the bottom of tank wall	
M_i^*	Overturning moment in impulsive mode at the base	
p	Maximum hydrodynamic pressure on wall	Refer Clause 4.10.2
p_{cb}	Convective hydrodynamic pressure on tank base	Refer Clause 4.9.2
p_{cw}	Convective hydrodynamic pressure on tank wall	
p_{ib}	Impulsive hydrodynamic pressure on tank base	Refer Clause 4.9.2
p_{iw}	Impulsive hydrodynamic pressure on tank wall	
p_v	Hydrodynamic pressure on tank wall due to vertical ground acceleration	
p_{ww}	Pressure on wall due to its inertia	
q	Uniformly distributed pressure on one wall of rectangular tank in the direction of ground motion	Refer Clause 4.3.1.2 and Figure C-2

q_i	Impulsive hydrodynamic force per unit length of wall
q_c	Convective hydrodynamic force per unit length of wall

Q_{cb}	Coefficient of convective pressure on tank base
Q_{cw}	Coefficient of convective pressure

PROVISIONS

on tank wall

Q_{ib} Coefficient of impulsive pressure on tank base

Q_{iw} Coefficient of impulsive pressure on tank wall

R Response reduction factor given in Table 2 of this code

(S_a/g) Average response acceleration coefficient as per IS 1893 (Part 1): 2002 and Clause 4.5 of this code

t Thickness of tank wall

t_b Thickness of base slab

T_c Time period of convective mode (in seconds)

T_i Time period of impulsive mode (in seconds)

V Total base shear

V_c Base shear in convective mode

V_i Base shear in impulsive mode

x Horizontal distance in the direction of seismic force, of a point on base slab from the reference axis at the center of tank

y Vertical distance of a point on tank wall from the bottom of tank wall

Z Seismic zone factor as per Table 2 of IS 1893 (Part 1): 2002

ρ Mass density of liquid

ρ_w Mass density of tank wall

ϕ Circumferential angle as described in Figure 8a

COMMENTARY

T Time period in seconds

V' Design base shear at the bottom of base slab/plate of ground supported tank

Refer Figure 8a

Refer Figure 8a

γ_s Density of soil

μ_c Convective bending moment coefficient

μ_i Impulsive bending moment coefficient

In SI Units, mass density will be in kg/m^3, while weight density will be in Newton N/m^3

Δ Deflection of center of gravity of tank when a lateral force of magnitude $(m_s+m_i)g$ is applied at the center of gravity of tank

1.

PROVISIONS

4. – Provisions for Seismic Design

4.1 - General

Hydrodynamic forces exerted by liquid on tank wall shall be considered in the analysis in addition to hydrostatic forces. These hydrodynamic forces are evaluated with the help of spring mass model of tanks.

4.2 - Spring Mass Model for Seismic Analysis

When a tank containing liquid vibrates, the liquid exerts impulsive and convective hydrodynamic pressure on the tank wall and the tank base in addition to the hydrostatic pressure. In order to include the effect of hydrodynamic pressure in the analysis, tank can be idealized by an equivalent spring mass model, which includes the effect of tank wall – liquid interaction. The parameters of this model depend on geometry of the tank and its flexibility.

COMMENTARY

C4.– Provisions for Seismic Design

C4.1 –

Dynamic analysis of liquid containing tank is a complex problem involving fluid-structure interaction. Based on numerous analytical, numerical, and experimental studies, simple spring mass models of tank-liquid system have been developed to evaluate hydrodynamic forces.

C4.2 – Spring Mass Model for Seismic Analysis

When a tank containing liquid with a free surface is subjected to horizontal earthquake ground motion, tank wall and liquid are subjected to horizontal acceleration. The liquid in the lower region of tank behaves like a mass that is rigidly connected to tank wall. This mass is termed as impulsive liquid mass which accelerates along with the wall and induces impulsive hydrodynamic pressure on tank wall and similarly on base. Liquid mass in the upper region of tank undergoes sloshing motion. This mass is termed as convective liquid mass and it exerts convective hydrodynamic pressure on tank wall and base. Thus, total liquid mass gets divided into two parts, i.e., impulsive mass and convective mass. In spring mass model of tank-liquid system, these two liquid masses are to be suitably represented.

A qualitative description of impulsive and convective hydrodynamic pressure distribution on tank wall and base is given in Figure C-1.

Sometimes, vertical columns and shaft are present inside the tank. These elements cause obstruction to sloshing motion of liquid. In the presence of such obstructions, impulsive and convective pressure distributions are likely to change. At present, no study is available to quantify effect of such obstructions on impulsive and convective pressures. However, it is reasonable to expect that due to presence of such obstructions, impulsive pressure will increase and connective pressure will decrease.

PROVISIONS

4.2.1 – Ground Supported Tank

4.2.1.1 –

Ground supported tanks can be idealized as spring-mass model shown in Figure 1. The impulsive mass of liquid, m_i is rigidly attached to tank wall at height h_i (or h_i^*). Similarly, convective mass, m_c is attached to the tank wall at height h_c (or h_c^*) by a spring of stiffness K_c.

COMMENTARY

C4.2.1 – Ground Supported Tank

C4.2.1.1 –

The spring mass model for ground supported tank is based on work of Housner (1963a).

In the spring mass model of tank, h_i is the height at which the resultant of impulsive hydrodynamic pressure on wall is located from the bottom of tank wall. On the other hand, h_i^* is the height at which the resultant of impulsive pressure on wall and base is located from the bottom of tank wall. Thus, if effect of base pressure is not considered, impulsive mass of liquid, m_i will act at a height of h_i and if effect of base pressure is considered, m_i will act at h_i^*. Heights h_i and h_i^*, are schematically described in Figures C-1a and C-1b.

Similarly, h_c, is the height at which resultant of convective pressure on wall is located from the bottom of tank wall, while, h_c^* is the height at which resultant of convective pressure on wall and base is located. Heights h_c and h_c^* are described in Figures C-1c and C-1d .

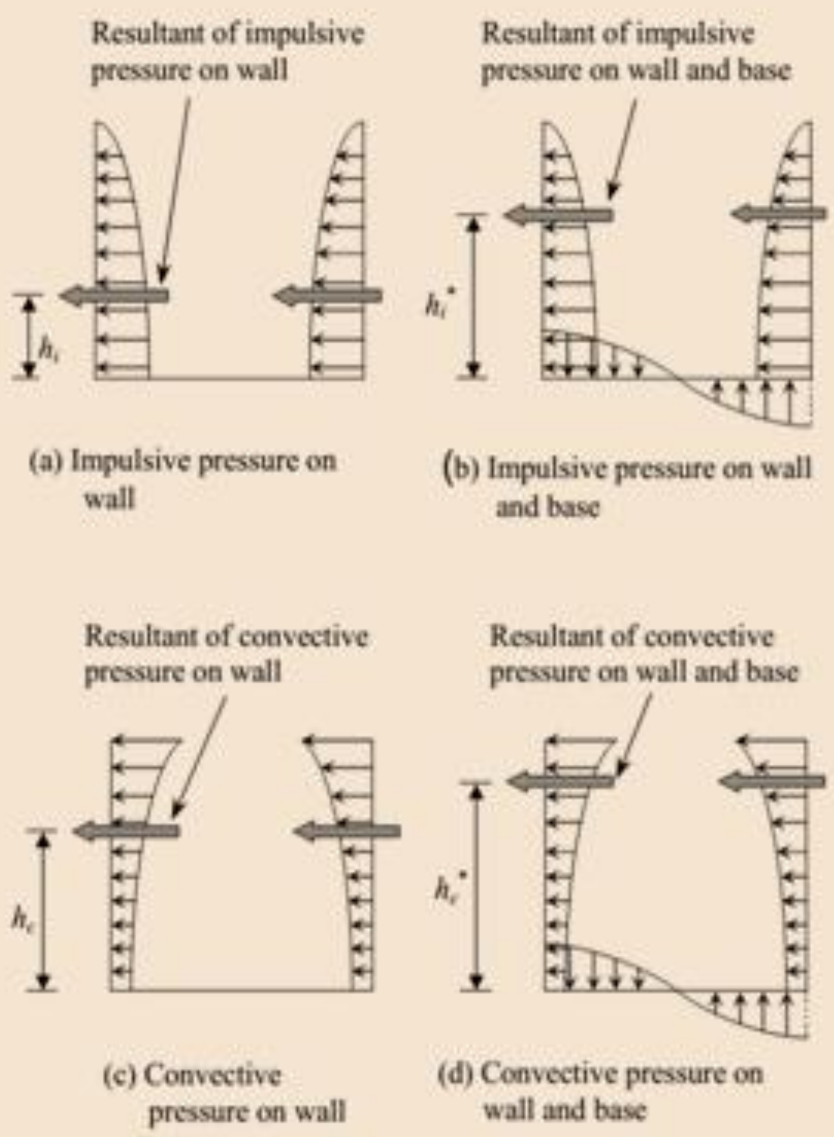


Figure C-1 Qualitative description of hydrodynamic pressure distribution on tank wall and base

PROVISIONS

4.2.1.2 – Circular and Rectangular Tank

For circular tanks, parameters m_i, m_c, h_i, h_i^*, h_c, h_c^* and K_c shall be obtained from Figure 2 and for rectangular tanks these parameters shall be obtained from Figure 3. h_i and h_c account for hydrodynamic pressure on the tank wall only. h_i^* and h_c^* account for hydrodynamic pressure on tank wall and the tank base. Hence, the value of h_i and h_c shall be used to calculate moment due to hydrodynamic pressure at the bottom of the tank wall. The value of h_i^* and h_c^* shall be used to calculate overturning moment at the base of tank.

COMMENTARY

C4.2.1.2 – Circular and Rectangular Tank

The parameters of spring mass model depend on tank geometry and were originally derived by Housner (1963a). The parameters shown in Figures 2 and 3 are slightly different from those given by Housner (1963a), and have been taken from ACI 350.3 (2001). Expressions for these parameters are given in Table C-1.

It may be mentioned that these parameters are for tanks with rigid walls. In the literature, spring-mass models for tanks with flexible walls are also available (Haroun and Housner (1981) and Veletsos (1984)). Generally, concrete tanks are considered as tanks with rigid wall; while steel tanks are considered as tanks with flexible wall. Spring mass models for tanks with flexible walls are more cumbersome to use. Moreover, difference in the parameters (m_i, m_c, h_i, h_i^*, h_c, h_c^* and K_c) obtained from rigid and flexible tank models is not substantial (Jaiswal et al. (2004b)). Hence in the present code, parameters corresponding to tanks with rigid wall are recommended for all types of tanks.

Further, flexibility of soil or elastic pads between wall and base do not have appreciable influence on these parameters.

It may also be noted that for certain values of *h/D* ratio, sum of impulsive mass (m_i) and convective mass (m_c) will not be equal to total mass (m) of liquid; however, the difference is usually small (2 to 3%). This difference is attributed to assumptions and approximations made in the derivation of these quantities.

One should also note that for shallow tanks, values of h_i^* and h_c^* can be greater than h (Refer Figures 2b and 3b) due to predominant contribution of hydrodynamic pressure on base.

If vertical columns and shaft are present inside the tank, then impulsive and convective masses will change. Though, no study is available to quantify effect of such obstructions, it is reasonable to expect that with the presence of such obstructions, impulsive mass will increase and convective mass will decrease. In absence of more detailed analysis of such tanks, as an approximation, an equivalent cylindrical tank of same height and actual water mass may be considered to obtain impulsive and convective masses.

PROVISIONS

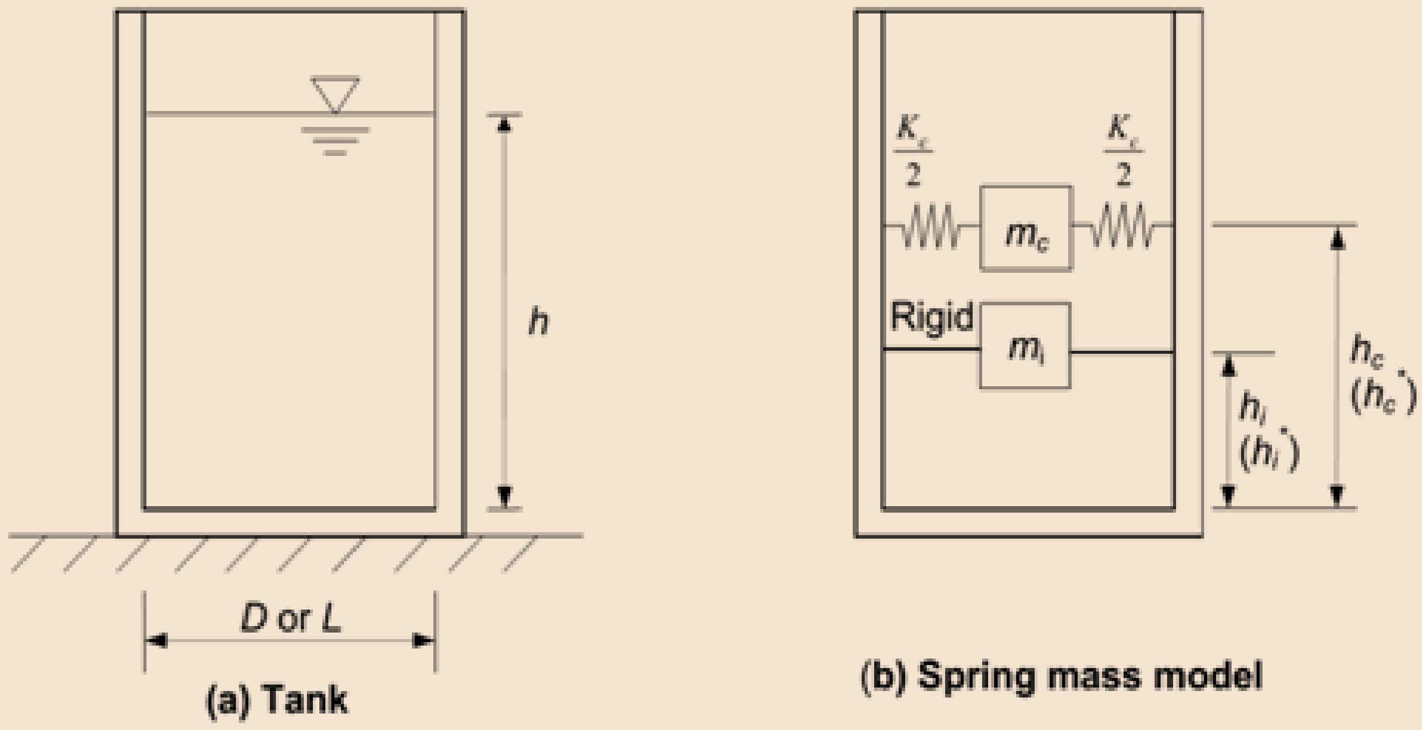


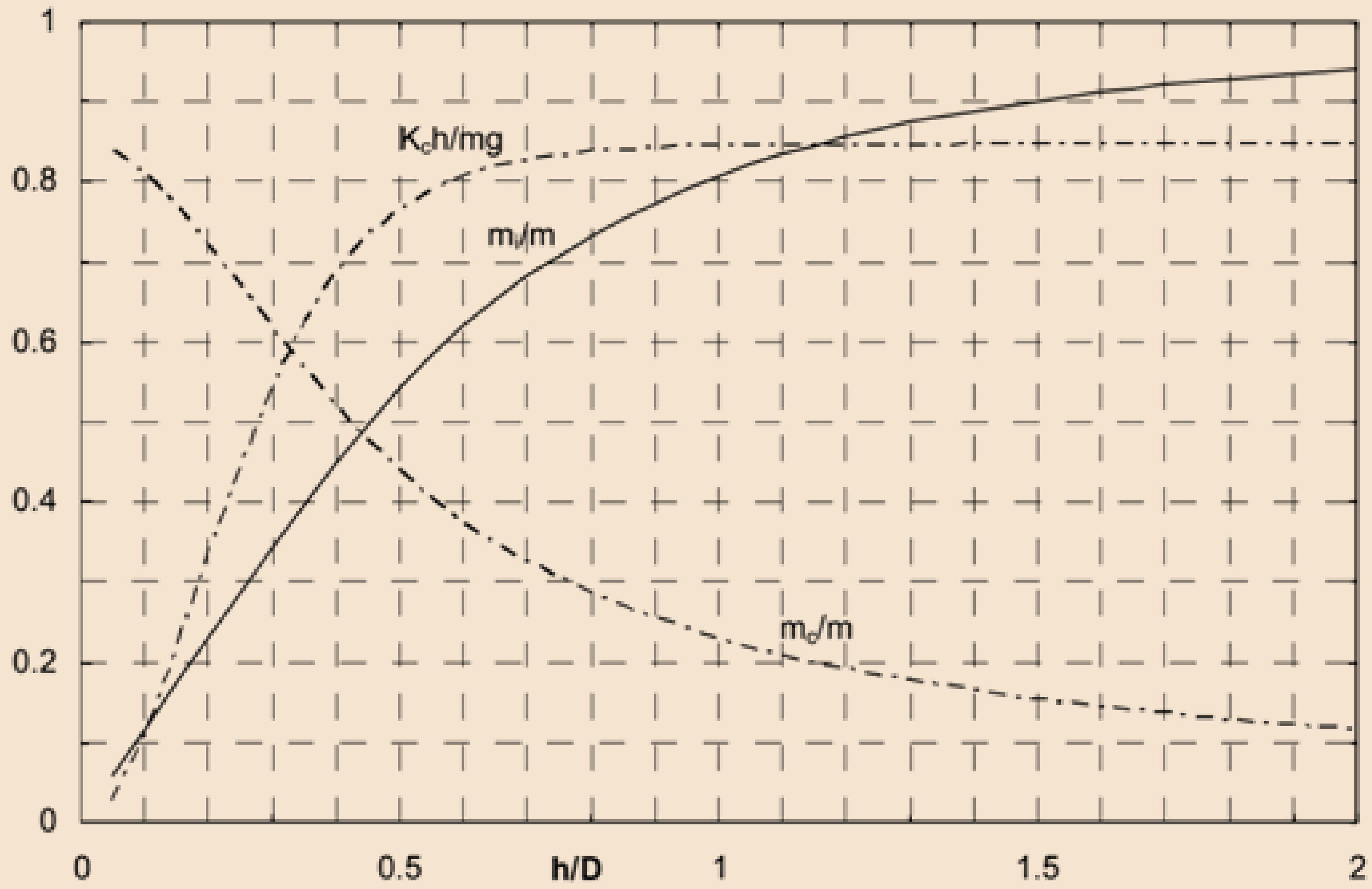
Figure 1 – Spring mass model for ground supported circular and rectangular tank

COMMENTARY

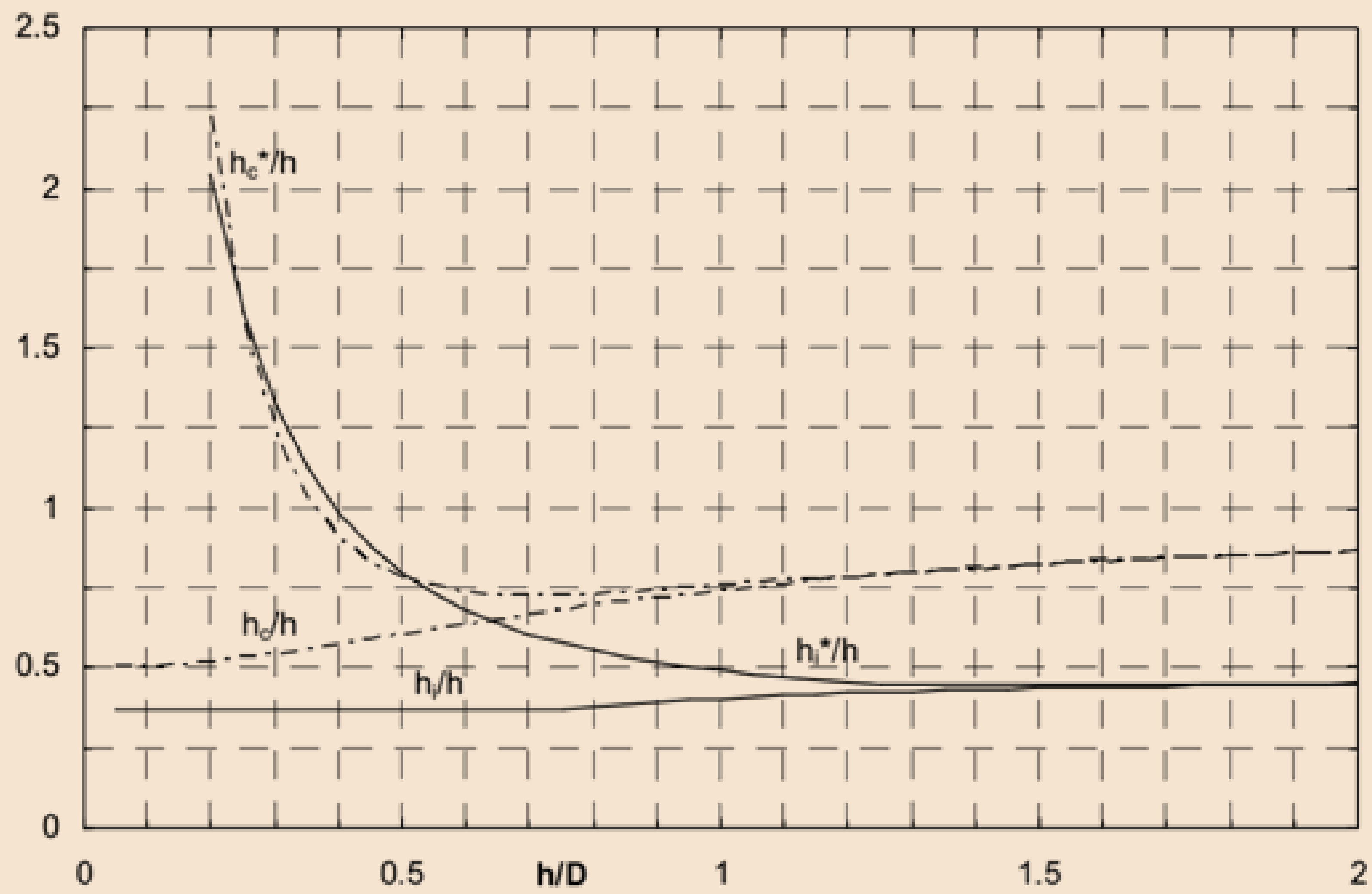
Table C 1 – Expression for parameters of spring mass model

Circular tank	Rectangular tank
$\frac{m_i}{m} = \frac{\tanh\left(0.866\frac{D}{h}\right)}{0.866\frac{D}{h}}$	$\frac{m_i}{m} = \frac{\tanh\left(0.866\frac{L}{h}\right)}{0.866\frac{L}{h}}$
$\frac{h_i}{h} = 0.375$ for $h/D \le 0.75$	$\frac{h_i}{h} = 0.375$ for $h/L \le 0.75$
$= 0.5 - \frac{0.09375}{h/D}$ for $h/D > 0.75$	$= 0.5 - \frac{0.09375}{h/L}$ for $h/L > 0.75$
$\frac{h_i^*}{h} = \frac{0.866\frac{D}{h}}{2\tanh\left(0.866\frac{D}{h}\right)} \quad 0.125$ for $h/D \le 1.33$	$\frac{h_i^*}{h} = \frac{0.866\frac{L}{h}}{2\tanh\left(0.866\frac{L}{h}\right)} - 0.125$ for $h/L \le 1.33$
$= 0.45$ for $h/D > 1.33$	$= 0.45$ for $h/L > 1.33$
$\frac{m_c}{m} = 0.23\frac{\tanh\left(3.68\frac{h}{D}\right)}{\frac{h}{D}}$	$\frac{m_c}{m} = 0.264\frac{\tanh\left(3.16\frac{h}{L}\right)}{\frac{h}{L}}$
$\frac{h_c}{h} = 1 - \frac{\cosh\left(3.68\frac{h}{D}\right) - 1.0}{3.68\frac{h}{D}\sinh\left(3.68\frac{h}{D}\right)}$	$\frac{h_c}{h} = 1 - \frac{\cosh\left(3.16\frac{h}{L}\right) - 1.0}{3.16\frac{h}{L}\sinh\left(3.16\frac{h}{L}\right)}$
$\frac{h_c^*}{h} = 1 - \frac{\cosh\left(3.68\frac{h}{D}\right) - 2.01}{3.68\frac{h}{D}\sinh\left(3.68\frac{h}{D}\right)}$	$\frac{h_c^*}{h} = 1 - \frac{\cosh\left(3.16\frac{h}{L}\right) - 2.01}{3.16\frac{h}{L}\sinh\left(3.16\frac{h}{L}\right)}$
$K_c = 0.836\frac{mg}{h}\tanh^2\left(3.68\frac{h}{D}\right)$	$K_c = 0.833\frac{mg}{h}\tanh^2\left(3.16\frac{h}{L}\right)$

PROVISIONS



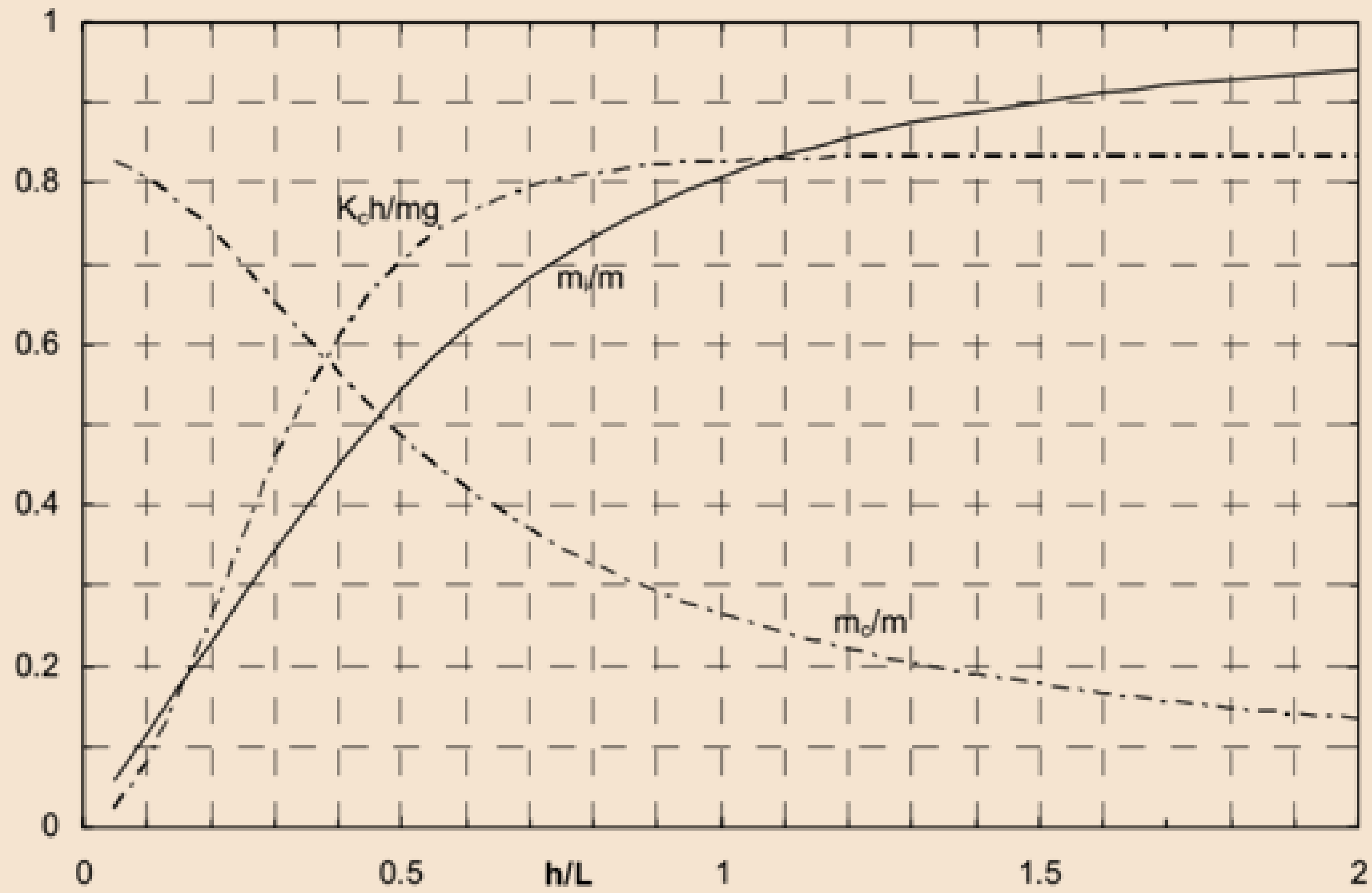
(a) Impulsive and convective mass and convective spring stiffness



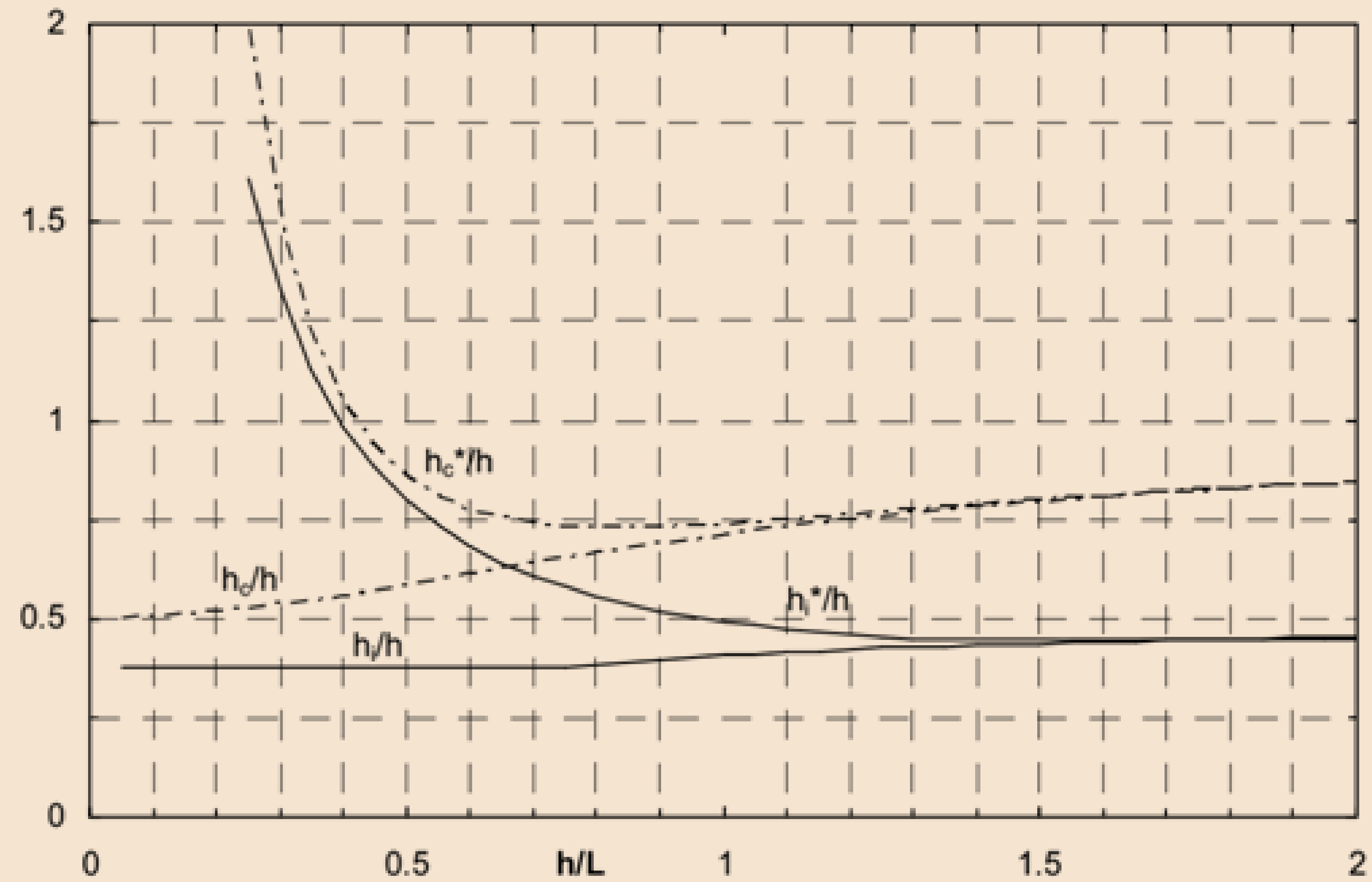
(b) Heights of impulsive and convective masses

Figure 2 – Parameters of the spring mass model for circular tank

PROVISIONS



(a) Impulsive and convective mass and convective spring stiffness



(b) Heights of impulsive and convective masses

Figure 3 – Parameters of the spring mass model for rectangular tank

PROVISIONS

4.2.3 –

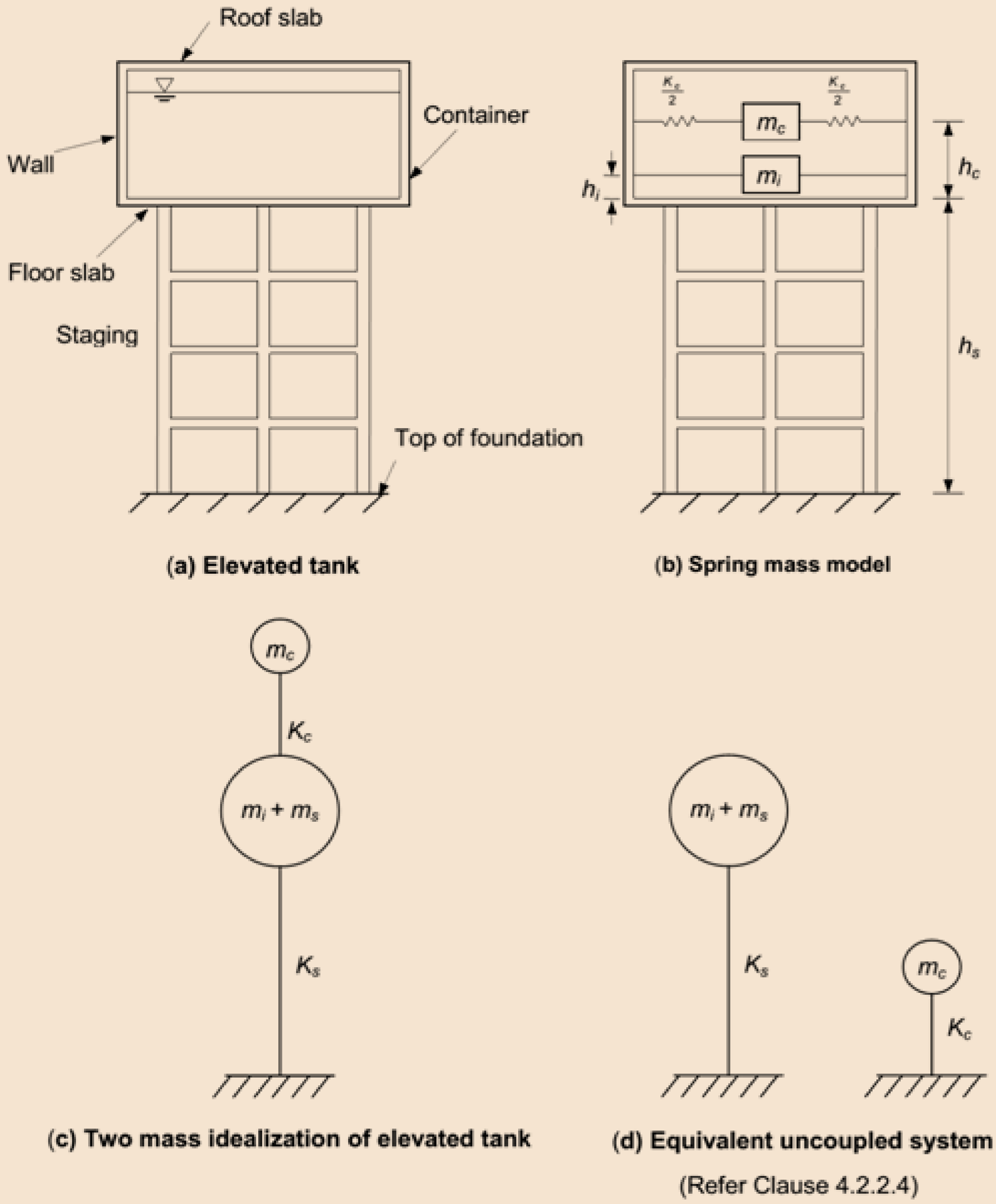
For tank shapes other than circular and rectangular (like intze, truncated conical shape), the value of h/D shall correspond to that of an equivalent circular tank of same volume and diameter equal to diameter of tank at top level of liquid; and m_i, m_c, h_i, h_i^*, h_c, h_c^* and K_c of equivalent circular tank shall be used.

COMMENTARY

C4.2.3 –

Parameters of spring mass models (i.e., m_i, m_c, h_i, h_i^*, h_c, h_c^* and K_c) are available for circular and rectangular tanks only. For tanks of other shapes, equivalent circular tank is to be considered. Joshi (2000) has shown that such an approach gives satisfactory results for intze tanks. Similarly, for tanks of truncated conical shape, Eurocode 8 (1998) has suggested equivalent circular tank approach.

PROVISIONS



(a) Elevated tank

(b) Spring mass model

(c) Two mass idealization of elevated tank

(d) Equivalent uncoupled system

(Refer Clause 4.2.2.4)

Figure 4 – Two mass idealization for elevated tank

PROVISIONS

4.3 – Time Period

4.3.1 – Impulsive Mode

4.3.1.1 – Ground Supported Circular Tank

For a ground supported circular tank, wherein wall is rigidly connected with the base slab (Figure 6a, 6b and 6c), time period of impulsive mode of vibration T_i, in seconds, is given by

$$T_i = C_i \frac{h\sqrt{\rho}}{\sqrt{t/D}\sqrt{E}}$$

where

C_i = Coefficient of time period for impulsive mode. Value of C_i can be obtained from Figure 5,

h = Maximum depth of liquid,

D = Inner diameter of circular tank,

t = Thickness of tank wall,

E = Modulus of elasticity of tank wall, and

ρ = Mass density of liquid.

NOTE: In some circular tanks, wall may have flexible connection with the base slab. (Different types of wall to base slab connections are described in Figure 6.) For tanks with flexible connections with base slab, time period evaluation may properly account for the flexibility of wall to base connection.

4.3.1.2 – Ground Supported Rectangular Tank

For a ground supported rectangular tank, wherein wall is rigidly connected with the base slab, time period of impulsive mode of vibration, T_i in seconds, is given by

COMMENTARY

C4.3 – Time Period

C4.3.1 – Impulsive Mode

C4.3.1.1 – Ground Supported Circular Tank

The coefficient C_i used in the expression of time period T_i and plotted in Figure 5, is given by

$$C_i = \left(\frac{1}{\sqrt{h/D}\left(0.46 - 0.3h/D + 0.067(h/D)^2\right)} \right)$$

The expression for the impulsive mode time period of circular tank is taken from Eurocode 8 (1998). Basically this expression was developed for roofless steel tank fixed at base and filled with water. However, this may also be used for other tank materials and fluids. Further, it may be mentioned that this expression is derived based on the assumption that tank mass is quite small compared to mass of fluid. This condition is usually satisfied by most of the tanks. More information on exact expression for time period of circular tank may be obtained from Veletsos (1984) and Natchigall et al. (2003).

In case of tanks with variable wall thickness (particularly, steel tanks with step variation of thickness), thickness of tank wall at 1/3rd height from the base should be used in the expression for impulsive time period.

Expression for T_i given in this section is applicable to only those circular tanks in which wall is rigidly attached to base slab. In some concrete tanks, wall is not rigidly attached to the base slab, and flexible pads are used between the wall and the base slab (Figure 6d to 6f). In such cases, flexibility of pads affects the impulsive mode time period. Various types of flexible connections between wall and base slab described in Figure 6 are taken from ACI 350.3 (2001), which provides more information on effect of flexible pads on impulsive mode time period.

C4.3.1.2–Ground Supported Rectangular Tank

Eurocode 8 (1998) and Preistley et al. (1986) also specify the same expression for obtaining time period of rectangular tank.

PROVISIONS

$$T_i = 2\pi\sqrt{\frac{d}{g}}$$

where

d = deflection of the tank wall on the vertical center-line at a height of $\bar{h}$, when loaded by uniformly distributed pressure of intensity q,

$$q = \frac{\left(\frac{m_i}{2} + \overline{m}_w\right)g}{Bh},$$

$$\bar{h} = \frac{\frac{m_i}{2}h_i + \overline{m}_w\frac{h}{2}}{\frac{m_i}{2} + \overline{m}_w},$$

$\overline{m}_w$ = Mass of one tank wall perpendicular to the direction of seismic force, and

B = Inside width of tank.

COMMENTARY

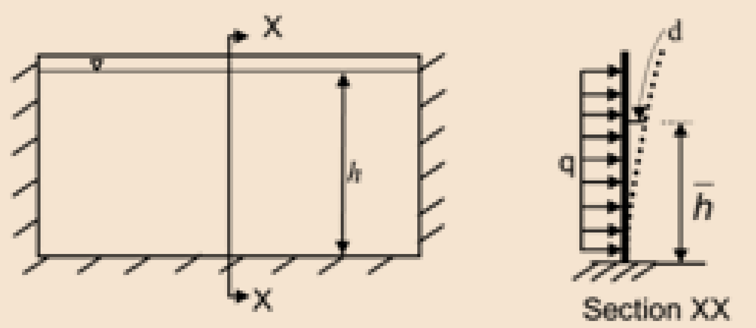
$\bar{h}$ is the height of combined center of gravity of half impulsive mass of liquid (m_i /2), and mass of one wall ($\overline{m}_w$).

For tanks without roof, deflection, d can be obtained by assuming wall to be free at top and fixed at three edges (Figures C-2a).

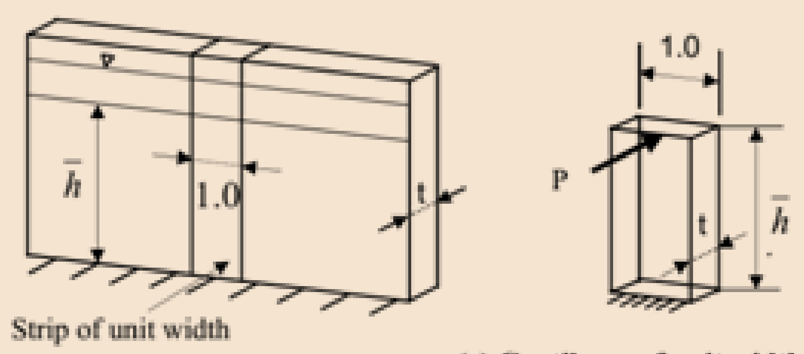
ACI 350.3 (2001) and NZS 3106 (1986) have suggested a simpler approach for obtaining deflection, d for tanks without roof. As per this approach, assuming that wall takes pressure q by cantilever action, one can find the deflection, d, by considering wall strip of unit width and height $\bar{h}$, which is subjected to concentrated load, $P = q\ h$ (Figures C-2b and C-2c). Thus, for a tank with wall of uniform thickness, one can obtain d as follows:

$$d = \frac{P(\bar{h})^3}{3EI_w}\ ;\ \text{where}\ I_w = \frac{1.0 \times t^3}{12}$$

The above approach will give quite accurate results for tanks with long walls (say, length greater than twice the height). For tanks with roofs and/or tanks in which walls are not very long, the deflection of wall shall be obtained using appropriate method.



(a) Rectangular tank wall subjected to uniformly distributed pressure



(b) Description of strip of wall **(c) Cantilever of unit width**

Figure C-2 Description of deflection d, of rectangular tank wall

PROVISIONS

4.3.1.3 – Elevated Tank

Time period of impulsive mode, T_i in seconds, is given by

$$T_i = 2\pi\sqrt{\frac{m_i + m_s}{K_s}}$$

where

m_s = mass of container and one-third mass of staging, and

K_s = lateral stiffness of staging.

Lateral stiffness of the staging is the horizontal force required to be applied at the center of gravity of the tank to cause a corresponding unit horizontal displacement.

> NOTE: The flexibility of bracing beam shall be considered in calculating the lateral stiffness, K_s of elevated moment-resisting frame type tank staging.

COMMENTARY

C4.3.1.3 – Elevated Tank

Time period of elevated tank can also be expressed as:

$$T_i = 2\pi\sqrt{\frac{\Delta}{g}}$$

where, Δ is deflection of center of gravity of tank when a lateral force of magnitude $(m_s + m_i)g$ is applied at the center of gravity of tank.

Center of gravity of tank can be approximated as combined center of mass of empty container and impulsive mass of liquid. The impulsive mass m_i acts at a height of h_i from top of floor slab.

For elevated tanks with moment resisting type frame staging, the lateral stiffness can be evaluated by computer analysis or by simple procedures (Sameer and Jain, 1992), or by established structural analysis method.

In the analysis of staging, due consideration shall be given to modeling of such parts as spiral staircase, which may cause eccentricity in otherwise symmetrical staging configuration.

For elevated tanks with shaft type staging, in addition to the effect of flexural deformation, the effect of shear deformation may be included while calculating the lateral stiffness of staging.

PROVISIONS

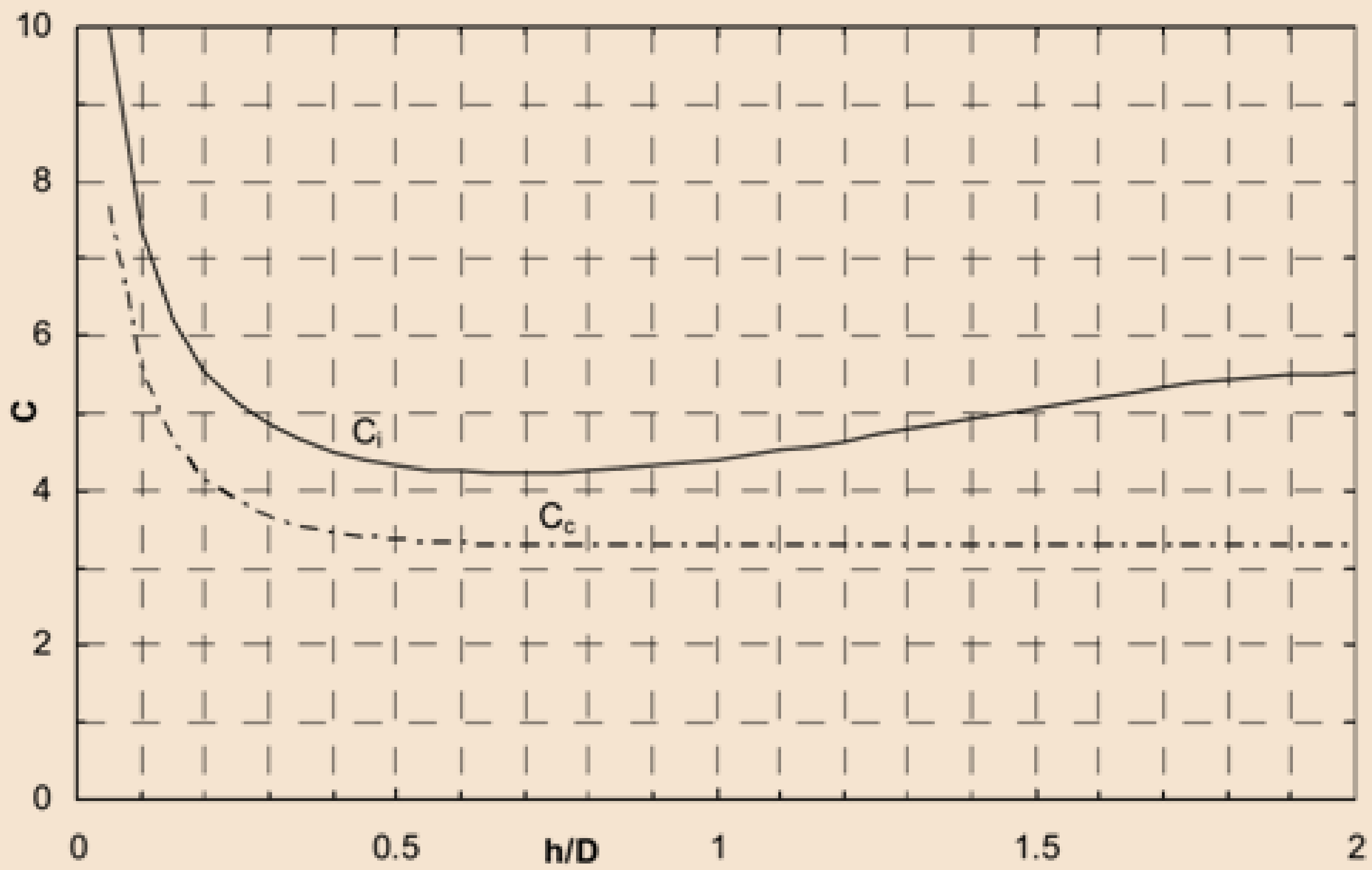


Figure 5 – Coefficient of impulsive (C_i) and convective (C_c) mode time period for circular tank

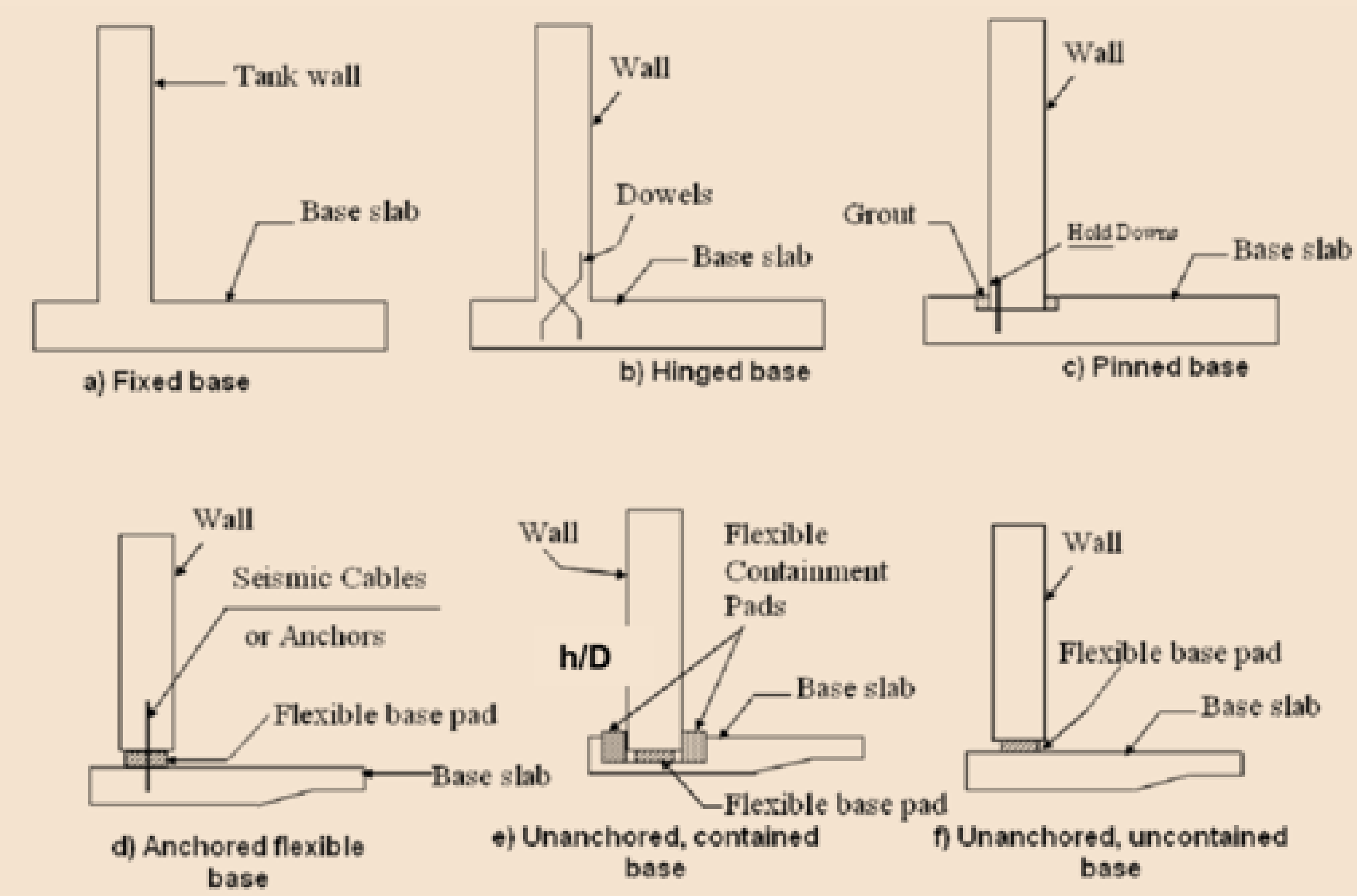


Figure 6 – Types of connections between tank wall and base slab

PROVISIONS

4.3.2 – Convective Mode

4.3.2.1 –

Time period of convective mode, in seconds, is given by

$$T_c = 2\pi \sqrt{\frac{m_c}{K_c}}$$

The values of m_c and K_c can be obtained from Figures 2a and 3a respectively, for circular and rectangular tanks.

4.3.2.2 –

Since the expressions for m_c and K_c are known, the expression for T_c can be alternatively expressed as:

(a) Circular Tank: Time period of convective mode, T_c in seconds, is given by

$$T_c = C_c \sqrt{D / g}$$

where

C_c = Coefficient of time period for convective mode. Value of C_c can be obtained from Figure 5, and

D = Inner diameter of tank.

(b) Rectangular Tank: Time period of convective mode of vibration, T_c in seconds, is given by

$$T_c = C_c \sqrt{L / g}$$

where

C_c = Coefficient of time period for convective mode. Value of C_c can be obtained from Figure 7, and

L = Inside length of tank parallel to the

COMMENTARY

C4.3.2 – Convective Mode

C4.3.2.2 –

Expressions given in Clause 4.3.2.1 and 4.3.2.2 are mathematically same. The expressions for convective mode time period of circular and rectangular tanks are taken from ACI 350.3 (2001), which are based on work of Housner (1963a). The coefficients C_c in the expressions for convective mode time period plotted in Figure 5 and 7 are given below:

(a) For *circular tank*:

$$C_c = \frac{2\pi}{\sqrt{3.68\, tanh\,(3.68h / D)}}$$

(b) *For rectangular tank:*

$$C_c = \frac{2\pi}{\sqrt{3.16\, tanh\,(3.16(h / L))}}$$

Convective mode time period expressions correspond to tanks with rigid wall. It is well established that flexibility of wall, elastic pads, and soil does not affect the convective mode time period.

For rectangular tank, L is the inside length of tank parallel to the direction of loading, as described in

PROVISIONS

direction of seismic force.

COMMENTARY

Figure C-3.

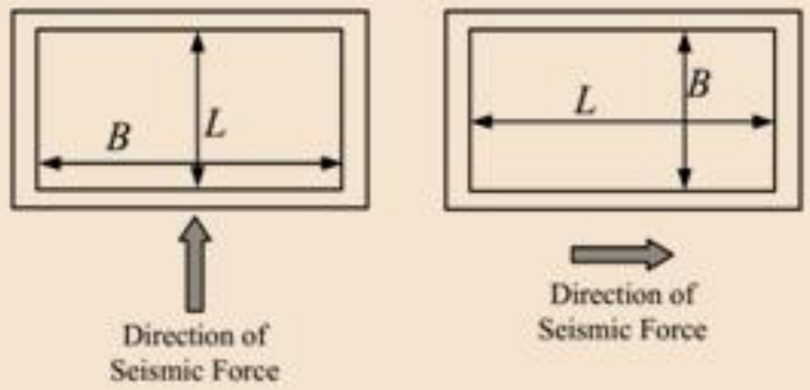


Figure C-3 Description of length, *L* and breadth, *B* of rectangular tank

4.3.3 –

For tanks resting on soft soil, effect of flexibility of soil may be considered while evaluating the time period. Generally, soil flexibility does not affect the convective mode time period. However, soil flexibility may affect impulsive mode time period.

C4.3.3 –

Soil structure interaction has two effects: Firstly, it elongates the time period of impulsive mode and secondly it increases the total damping of the system. Increase in damping is mainly due to radial damping effect of soil media. A simple but approximate approach to obtain the time period of impulsive mode and damping of tank-soil system is provided by Veletsos (1984). This simple approach has been used in Eurocode 8 (1998) and Priestley et al. (1986).

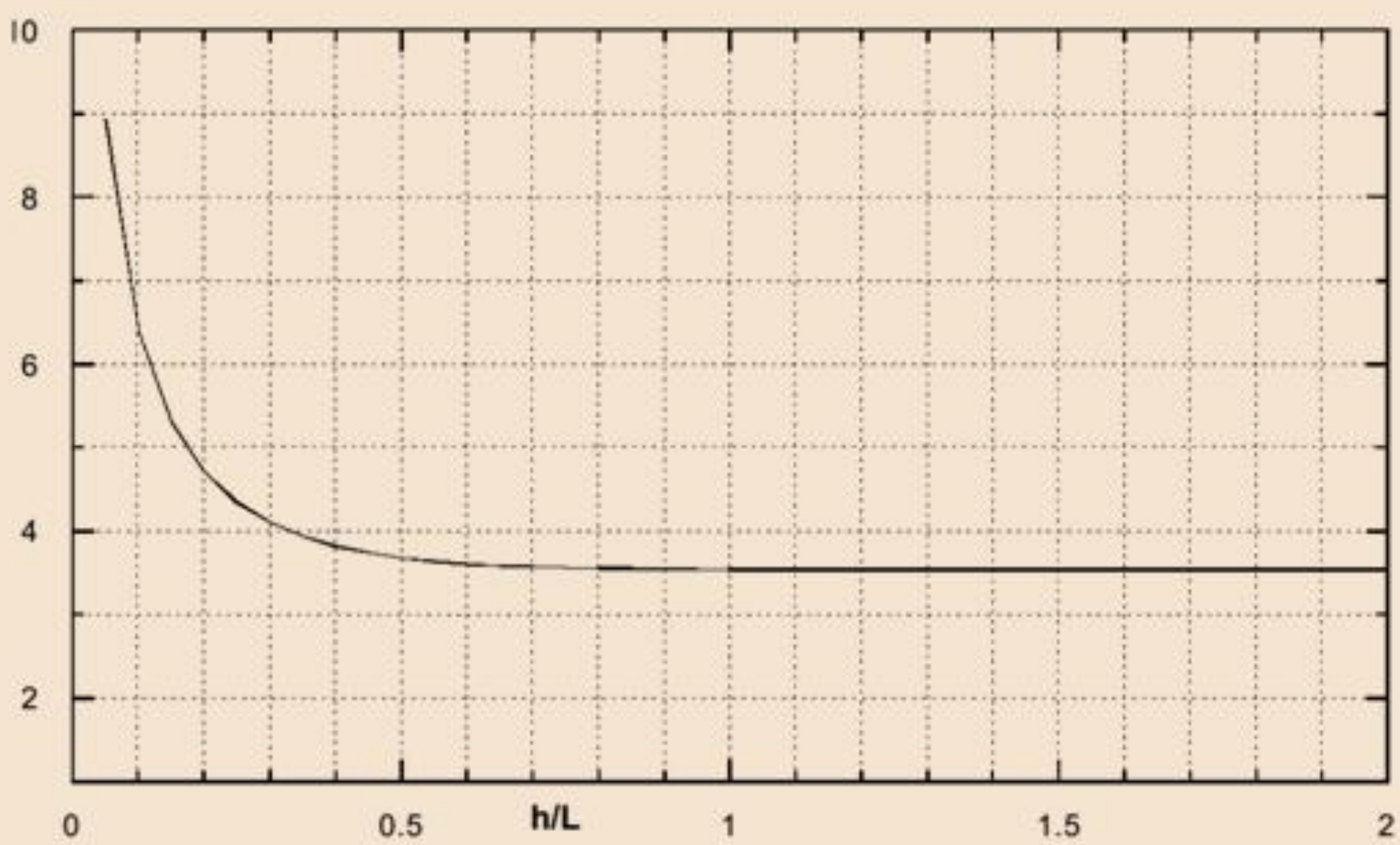


Figure 7 – Coefficient of convective mode time period (C_c) for rectangular tank

PROVISIONS

4.4 – Damping

Damping in the convective mode for all types of liquids and for all types of tanks shall be taken as 0.5% of the critical.

Damping in the impulsive mode shall be taken as 2% of the critical for steel tanks and 5% of the critical for concrete or masonry tanks.

4.5 – Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient, A_h shall be obtained by the following expression, subject to Clauses 4.5.1 to 4.5.4

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g}$$

where

Z = Zone factor given in Table 2 of IS 1893 (Part 1): 2002,

I = Importance factor given in Table 1 of this guideline,

R = Response reduction factor given in Table 2 of this guideline, and

S_a/g = Average response acceleration coefficient as given by Figure 2 and Table 3 of IS 1893(Part 1): 2002 and subject to Clauses 4.5.1 to 4.5.4 of this guideline.

COMMENTARY

C4.4 – Damping

For convective mode damping of 0.5% is used in most of the international codes.

C4.5 – Design Horizontal Seismic Coefficient

Importance factor (I), is meant to ensure a better seismic performance of important and critical tanks. Its value depends on functional need, consequences of failure, and post earthquake utility of the tank.

In this guideline, liquid containing tanks are put in three categories and importance factor to each category is assigned (Table 1). Highest value of I =1.75 is assigned to tanks used for storing hazardous materials. Since release of these materials can be harmful to human life, the highest value of I is assigned to these tanks. For tanks used in water distribution systems, value of I is kept as 1.5, which is same as value of I assigned to hospital, telephone exchange, and fire station buildings in IS 1893 (Part 1):2002. Less important tanks are assigned I = 1.0.

Response reduction factor (R), represents ratio of maximum seismic force on a structure during specified ground motion if it were to remain elastic to the design seismic force. Thus, actual seismic forces are reduced by a factor R to obtain design forces. This reduction depends on overstrength, redundancy, and ductility of structure. Generally, liquid containing tanks posses low overstrength, redundancy, and ductility as compared to buildings. In buildings, non structural components substantially contribute to overstrength; in tanks, such non structural components are not present. Buildings with frame type structures have high redundancy; ground supported tanks and elevated tanks with shaft type staging have comparatively low redundancy. Moreover, due to presence of non structural elements like masonry walls, energy absorbing capacity of buildings is much higher than that of tanks. Based on these considerations, value of R for tanks needs to be lower than that for buildings. All the international codes specify much lower values of R for tanks than those for buildings. As

PROVISIONS

Table 1 – Importance factor, *I*

Type of liquid storage tank	*I*
Tanks used for storing drinking water, non-volatile material, low inflammable petrochemicals etc. and intended for emergency services such as fire fighting services. Tanks of post earthquake importance.	1.5
All other tanks with no risk to life and with negligible consequences to environment, society and economy.	1.0

Note- Values of importance factor, I given in IS 1893 (Part 4) may be used where appropriate.

COMMENTARY

an example, values of R used in IBC 2000 are shown in Table C-2. It is seen that for a building with special moment resisting frame value of R is 8.0 whereas, for an elevated tank on frame type staging (i.e., braced legs), value of R is 3.0. Further, it may also be noted that value of R for tanks varies from 3.0 to 1.5.

Values of R given in the present guideline (Table 2) are based on studies of Jaiswal et al. (2004a, 2004b). In this study, an exhaustive review of response reduction factors used in various international codes is presented. In Table 2, the highest value of R is 2.5 and lowest value is 1.3. The rationale behind these values of R can be seen from Figures C-4a and C-4b.

In Figure C-4a, base shear coefficients (i.e., ratio of lateral seismic force to weight) obtained from IBC 2000 and IS 1893 (Part 1):2002 is compared for a building with special moment resisting frame. This comparison is done for the most severe seismic zone of IBC 2000 and IS 1893 (Part 1):2002. It is seen that base shear coefficient from IS 1893 (Part 1):2002 and IBC 2000 compare well, particularly up to time period of 1.7 sec.

In Figure C-4b, base shear coefficient for tanks is compared. This comparison is done for the highest as well as lowest value of R from IBC 2000 and present code. It is seen that base shear coefficient match well for highest and lowest value of R. Thus, the specified values of R are quite reasonable and in line with international practices.

Elevated tanks are inverted pendulum type structures and hence, moment resisting frames being used in staging of these tanks are assigned much smaller R values than moment resisting frames of building and industrial frames. For elevated tanks on frame type staging, response reduction factor is R = 2.5 and for elevated tanks on RC shaft, R = 1.8. Lower value of R for RC shaft is due to its low redundancy and poor ductility (Zahn, 1999; Rai 2002).

PROVISIONS

Table 2 – Response reduction factor, *R*

Type of tank	*R*
Elevated tank	
Tank supported on masonry shaft	
a) Masonry shaft reinforced with horizontal bands *	1.3
b) Masonry shaft reinforced with horizontal bands and vertical bars at corners and jambs of openings	1.5
Tank supported on RC shaft	
RC shaft with two curtains of reinforcement, each having horizontal and vertical reinforcement	1.8
Tank supported on RC frame#	
a) Frame not conforming to ductile detailing, i.e., ordinary moment resisting frame (OMRF)	1.8
b) Frame conforming to ductile detailing, i.e., special moment resisting frame (SMRF)	2.5
Tank supported on steel frame#	2.5
Ground supported tank	
Masonry tank	
a) Masonry wall reinforced with horizontal bands*	1.3
b) Masonry wall reinforced with horizontal bands and vertical bars at corners and jambs of openings	1.5
RC / prestressed tank	
a) Fixed or hinged/pinned base tank (Figures 6a, 6b, 6c)	2.0
b) Anchored flexible base tank (Figure 6d)	2.5
c) Unanchored contained or uncontained tank (Figures 6e, 6f)	1.5
Steel tank	
a) Unanchored base	2.0
b) Anchored base	2.5
Underground RC and steel tank+	4.0

These R values are meant for liquid retaining tanks on frame type staging which are inverted pendulum type structures. These R values shall not be misunderstood for those given in other parts of IS 1893 for building and industrial frames.

* These tanks are not allowed in seismic zones IV and V.

+ For partially buried tanks, values of *R* can be interpolated between ground supported and underground tanks based on depth of embedment.

COMMENTARY

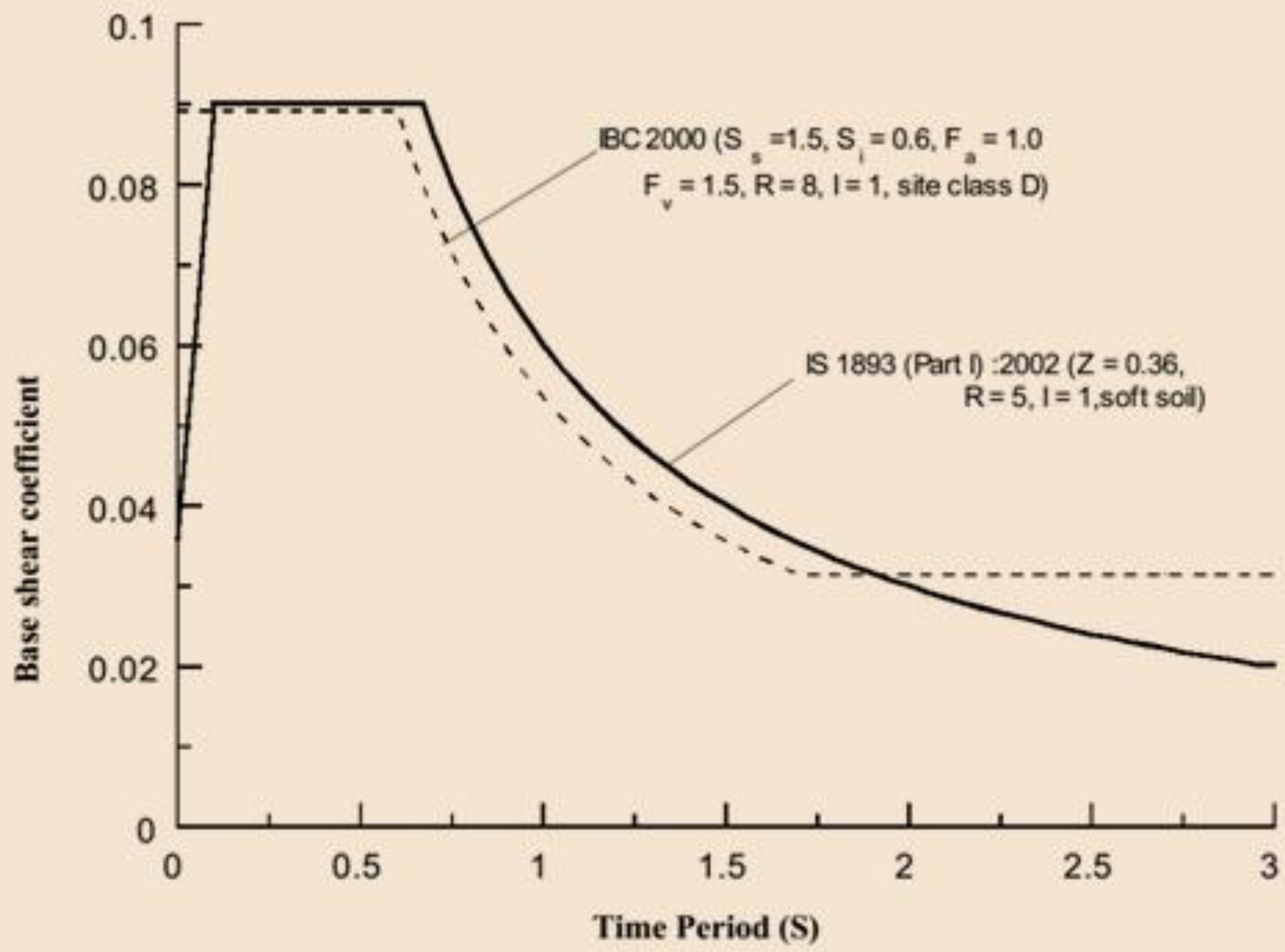


Figure C-4a Comparison of base shear coefficient obtained from IBC 2000 and IS 1893 (Part 1):2002, for a building with special moment resisting frame. IBC values are divided by 1.4 to bring them to working stress level (From Jaiswal et. al., 2004a)

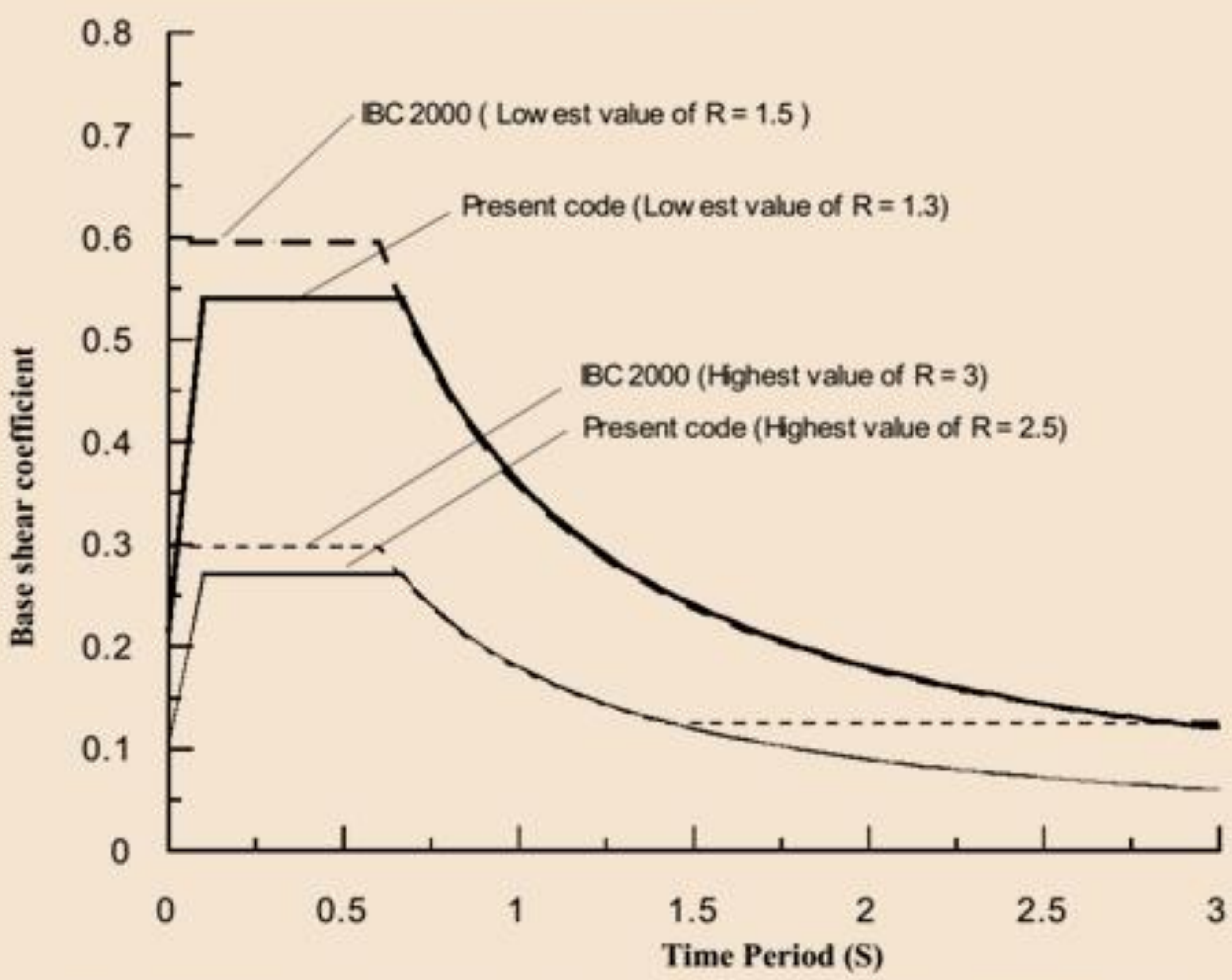


Figure C-4b Comparison of base shear coefficient obtained from IBC 2000 and present code, for tanks with highest and lowest values of R. (From Jaiswal et. al., 2004a)

PROVISIONS

COMMENTARY

Table C-2 Values of response reduction factor used in IBC 2000

Type of structure	R
Building with special reinforced concrete moment resisting concrete frames	8.0
Building with intermediate reinforced concrete moment resisting concrete frames	5.0
Building with ordinary reinforced concrete moment resisting concrete frames	3.0
Building with special steel concentrically braced frames	8.0
Elevated tanks supported on braced/unbraced legs	3.0
Elevated tanks supported on single pedestal	2.0
Tanks supported on structural towers similar to buildings	3.0
Flat bottom ground supported anchored steel tanks	3.0
Flat bottom ground supported unanchored steel tanks	2.5
Reinforced or prestressed concrete tanks with anchored flexible base	3.0
Reinforced or prestressed concrete tanks with reinforced nonsliding base	2.0
Reinforced or prestressed concrete tanks with unanchored and unconstrained flexible base	1.5

4.5.1 –

Design horizontal seismic coefficient, A_h will be calculated separately for impulsive $(A_h)_i$, and convective $(A_h)_c$ modes.

C4.5.1 –

The values of R, given in Table 2 of this code, are applicable to design horizontal seismic coefficient of impulsive as well as convective mode.

It may be noted that amongst various international codes, AWWA D-100, AWWA D-103 and AWWA D-115 use same value of R for impulsive and convective modes, whereas, ACI 350.3 and Eurocode 8 suggest value of R =1 for convective mode. The issue of value of R for convective component is still being debated by researchers and hence to retain the simplicity in the analysis, in the present provision, same value of R have been proposed for impulsive and convective components.

PROVISIONS

4.5.2 -

If time period is less than 0.1 second, the value of S_a/g shall be taken as 2.5 for 5% damping and be multiplied with appropriate factor, for other damping.

4.5.3 –

For time periods greater than four seconds, the value of S_a/g shall be obtained using the same expression which is applicable upto time period of four seconds.

4.5.4 -

Value of multiplying factor for 0.5% damping shall be taken as 1.75.

COMMENTARY

C4.5.3 –

Clauses 4.5.2 and 4.5.3, effectively imply response acceleration coefficient (S_a/g) as

For hard soil sites

$$S_a/g = 2.5 \quad \text{for } T < 0.4$$
$$= 1.0/T \quad \text{for } T \geq 0.4$$

For medium soil sites

$$S_a/g = 2.5 \quad \text{for } T < 0.55$$
$$= 1.36/T \quad \text{for } T \geq 0.55$$

For soft soil sites

$$S_a/g = 2.5 \quad \text{for } T < 0.67$$
$$= 1.67/T \quad \text{for } T \geq 0.67$$

C4.5.4 –

Table 3 of IS 1893 (Part 1): 2002 gives values of multiplying factors for 0% and 2% damping, and value for 0.5% damping is not given. One can not linearly interpolate the values of multiplying factors because acceleration spectrum values vary as a logarithmic function of damping (Newmark and Hall, 1982).

In Eurocode 8 (1998), value of multiplying factor is taken as 1.673 and as per ACI 350.3 and FEMA 368, this value is 1.5.

PROVISIONS

4.6 - Base Shear

4.6.1 - Ground Supported Tank

Base shear in impulsive mode, at the bottom of tank wall is given by

$$V_i = (A_h)_i\,(m_i + m_w + m_t)g$$

and base shear in convective mode is given by

$$V_c = (A_h)_c\, m_c\, g$$

where

$(A_h)_i$ = Design horizontal seismic coefficient for impulsive mode,

$(A_h)_c$ = Design horizontal seismic coefficient for convective mode,

m_i = Impulsive mass of water

m_w = Mass of tank wall

m_t = Mass of roof slab, and

g = Acceleration due to gravity.

4.6.2 – Elevated Tank

Base shear in impulsive mode, just above the base of staging (i.e. at the top of footing of staging) is given by

$$V_i = (A_h)_i\,(m_i + m_s)g$$

and base shear in convective mode is given by

$$V_c = (A_h)_c\, m_c\, g$$

where

m_s = Mass of container and one-third mass of staging.

4.6.3 –

Total base shear V, can be obtained by combining the base shear in impulsive and convective mode through Square root of Sum of Squares (SRSS) rule and is given as follows

$$V = \sqrt{V_i^2 + V_c^2}$$

COMMENTARY

C4.6 – Base Shear

C4.6.1 – Ground Supported Tank

Live load on roof slab of tank is generally neglected for seismic load computations. However, in some ground supported tanks, roof slab may be used as storage space. In such cases, suitable percentage of live load should be added in the mass of roof slab, m_t.

For concrete/masonry tanks, mass of wall and base slab may be evaluated using wet density of concrete/masonry.

For ground supported tanks, to obtain base shear at the bottom of base slab/plate, shear due to mass of base slab/plate shall be included. If the base shear at the bottom of tank wall is V then, base shear at the bottom of base slab, V', will be given by

$$V' = V + (A_h)_i\, m_b$$

where, m_b is mass of base slab/plate.

C4.6.2 – Elevated Tank

Clause 4.6.2 gives shear at the base of staging. Base shear at the bottom of tank wall can be obtained from Clause 4.6.1.

C4.6.3 –

Except Eurocode 8 (1998) all international codes use SRSS rule to combine response from impulsive and convective mode. In Eurocode 8 (1998) absolute summation rule is used, which is based on work of Malhotra (2000). The basis for absolute summation is that the convective mode time period may be several times the impulsive mode period, and hence, peak response of

PROVISIONS

COMMENTARY

impulsive mode will occur simultaneously when convective mode response is near its peak. However, recently through a numerical simulation for a large number of tanks, Malhotra (2004) showed that SRSS rule gives better results than absolute summation rule.

4.7 – Base Moment

4.7.1 – Ground Supported Tank

4.7.1.1 –

Bending moment in impulsive mode, at the bottom of wall is given by

$$M_i = (A_h)_i (m_i h_i + m_w h_w + m_t h_t) g$$

and bending moment in convective mode is given by

$$M_c = (A_h)_c m_c h_c g$$

where

h_w = Height of center of gravity of wall mass, and

h_t = Height of center of gravity of roof mass.

4.7.1.2 –

Overturning moment in impulsive mode to be used for checking the tank stability at the bottom of base slab/plate is given by

$$M_i^* = (A_h)_i \begin{bmatrix} m_i(h_i^* + t_b) + m_w(h_w + t_b) + \\ m_t(h_t + t_b) + m_b t_b / 2 \end{bmatrix} g$$

and overturning moment in convective mode is given by

C4.7 – Base Moment

C4.7.1 – Ground Supported Tank

C4.7.1.1 –

For obtaining bending moment at the bottom of tank wall, effect of hydrodynamic pressure on wall is considered. Hence, m_i and m_c are considered to be located at heights h_i and h_c, which are explained in Figures C-1a and C-1c and Clause 4.2.1.1.

Heights, h_i and h_c are measured from top of the base slab or bottom of wall.

Sometimes it may be of interest to obtain bending moment at the intermediate height of tank wall. The bending moment at height, y from bottom will depend only on hydrodynamic pressure and wall mass above that height. Following Malhotra (2004), bending moment at any height y from the bottom of wall will be given by

$$M_i = (A_h)_i \begin{bmatrix} m_i\, h_i \mu_i + m_w\, h_w (1 - y/h)^2 / 2 \\ + m_t\, h_t (1 - y/h) \end{bmatrix} g$$

$$M_c = (A_h)_c m_c\, h_c \mu_c\, g$$

The value of μ_i and μ_c can be obtained from Figure C-5.

Second term in the expression of M_i is obtained by considering tank wall of uniform thickness.

C4.7.1.2 –

For obtaining overturning moment at the base of tank, hydrodynamic pressure on tank wall as well as tank base is considered. Hence, m_i and m_c are considered to be located at h_i^*, and h_c^*, which are described in Figures C-1b and C-1d.

PROVISIONS

COMMENTARY

$M^*_c = (A_h)_c \, m_c (h_c^* + t_b) g$

where

m_b = mass of base slab/plate, and

t_b = thickness of base slab/plate.

4.7.2 – Elevated Tank

Overturning moment in impulsive mode, at the base of the staging is given by

$M_i^* = (A_h)_i \left[m_i (h_i^* + h_s) + m_s \, h_{cg} \right] g$

and overturning moment in convective mode is given by

$M_c^* = (A_h)_c \, m_c (h_c^* + h_s) g$

where

h_s = Structural height of staging, measured from top of footing of staging to the bottom of tank wall, and

h_{cg} = Height of center of gravity of empty container, measured from top of footing.

C4.7.2 – Elevated Tank

Structural mass m_s, which includes mass of empty container and one-third mass of staging is considered to be acting at the center of gravity of empty container.

Base of staging may be considered at the top of footing.

4.7.3 –

Total moment shall be obtained by combining the moment in impulsive and convective modes through Square of Sum of Squares (SRSS) and is given as follows

$$M = \sqrt{M_i^2 + M_c^2}$$

$$M^* = \sqrt{M^{*2}_i + M^{*2}_c}$$

C4.7.3 –

See commentary of Clause 4.6.3

4.7.4 –

For elevated tanks, the design shall be worked out for tank empty and tank full conditions.

C4.7.4 –

For tank empty condition, convective mode of vibration will not be generated. Thus, empty elevated tank has to be analyzed as a single degree of freedom system wherein, mass of empty container and one-third mass of staging must be considered.

As such, ground supported tanks shall also be analysed for tank empty condition. However, being very rigid, it is unlikely that tank empty condition will become critical for ground supported tanks.

PROVISIONS

4.8 – Direction of Seismic Force

4.8.1 –

Ground supported rectangular tanks shall be analyzed for horizontal earthquake force acting non-concurrently along each of the horizontal axes of the tank for evaluating forces on tank walls.

4.8.2 –

For elevated tanks, staging components should be designed for the critical direction of seismic force. Different components of staging may have different critical directions.

4.8.3 –

As an alternative to 4.8.2, staging components can be designed for either of the following load combination rules:

i) 100% + 30% Rule:

$\pm EL_x \pm 0.3\ EL_y$ and $\pm 0.3\ EL_x \pm EL_y$

ii) SRSS Rule:

$$\sqrt{EL_x^2 + EL_y^2}$$

Where, EL_x is response quantity due to earthquake load applied in x-direction and EL_y is response quantity due to earthquake load applied in y-direction.

COMMENTARY

C4.8 – Direction of Seismic Force

C4.8.1 –

Base shear and stresses in a particular wall shall be based on the analysis for earthquake loading in the direction perpendicular to that wall.

C4.8.2 –

For elevated tanks supported on frame type staging, the design of staging members should be for the most critical direction of horizontal base acceleration. For a staging consisting of four columns, horizontal acceleration in diagonal direction (i.e. 45° to X-direction) turns out to be most critical for axial force in columns. For brace beam, most critical direction of loading is along the length of the brace beam.

Sameer and Jain (1994) have discussed in detail the critical direction of horizontal base acceleration for frame type staging.

For some typical frame type staging configurations, critical direction of seismic force is described in Figure C-6.

C4.8.3 –

100% + 30% rule implies following eight load combinations:

$(EL_x + 0.3\ EL_y)$;	$(EL_x - 0.3\ EL_y)$
$-(EL_x + 0.3\ EL_y)$;	$-(EL_x - 0.3\ EL_y)$
$(0.3EL_x + EL_y)$;	$(0.3EL_x - EL_y)$
$-(0.3EL_x + EL_y)$;	$-(0.3EL_x - EL_y)$

COMMENTARY

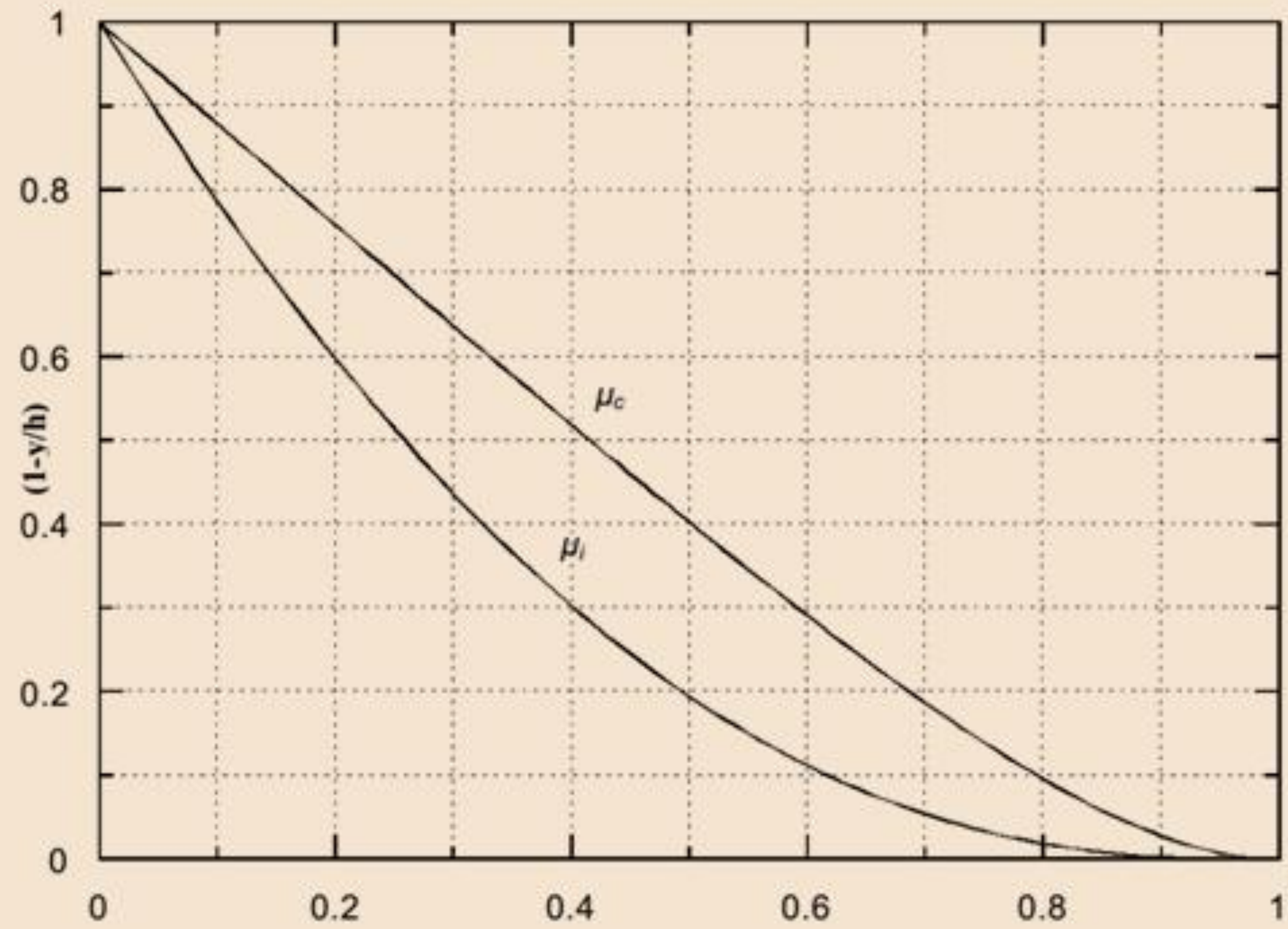


Fig. C- 5 Variation of impulsive and convective bending moment coefficients with height (From Malhotra, 2004)

PROVISIONS

COMMENTARY

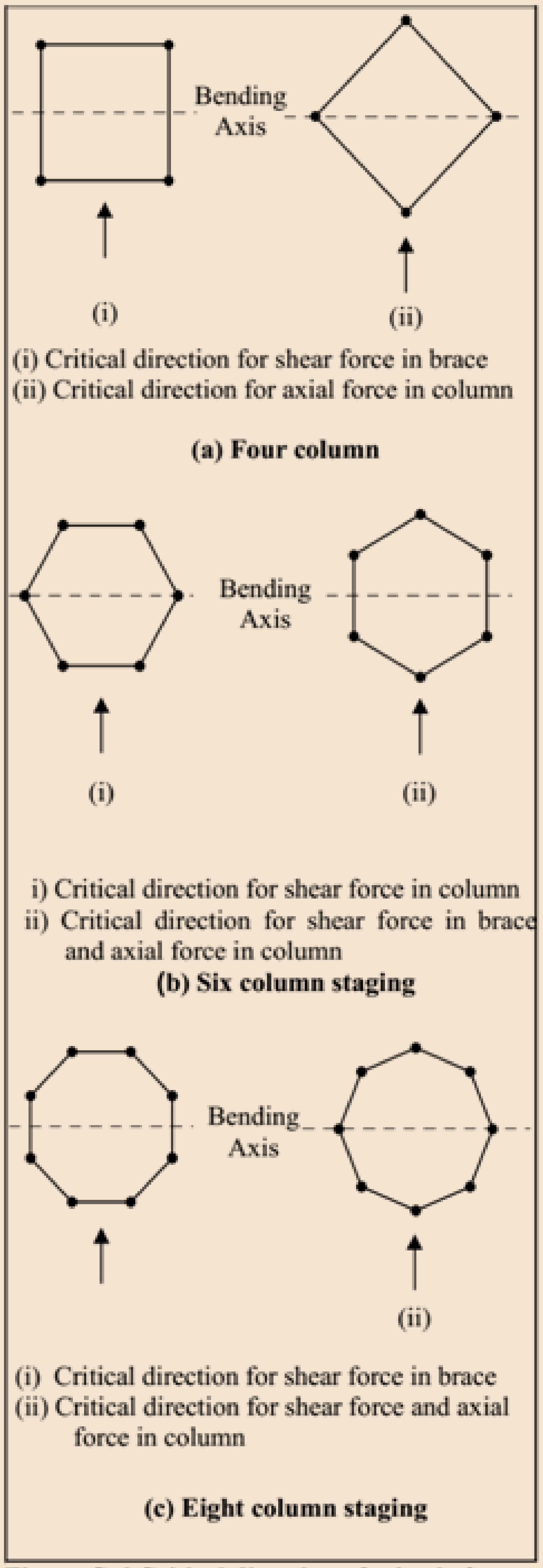


Figure C-6 Critical direction of seismic force for typical frame type staging profiles

PROVISIONS

4.9 – Hydrodynamic Pressure

During lateral base excitation, tank wall is subjected to lateral hydrodynamic pressure and tank base is subjected to hydrodynamic pressure in vertical direction (Figure C-1).

4.9.1 – Impulsive Hydrodynamic Pressure

The impulsive hydrodynamic pressure exerted by the liquid on the tank wall and base is given by

(a) For Circular Tank (Figure 8a)

Lateral hydrodynamic impulsive pressure on the wall, p_{iw} , is given by

$$p_{iw} = Q_{iw}(y)\,(A_h)_i \rho g h \;\cos\phi$$

$$Q_{iw}(y) = 0.866\left[1-\left(\frac{y}{h}\right)^2\right]\tanh\left(0.866\,\frac{D}{h}\right)$$

where

ρ = Mass density of liquid,

ϕ = Circumferential angle, and

y = Vertical distance of a point on tank wall from the bottom of tank wall.

Coefficient of impulsive hydrodynamic pressure on wall, $Q_{iw}(y)$ can also be obtained from Figure 9a.

Impulsive hydrodynamic pressure in vertical direction, on base slab (y = 0) on a strip of length l', is given by

$$p_{ib} = 0.866\,(A_h)_i\,\rho\, g\, h\,\frac{\sinh\left(1.732\,\frac{x}{h}\right)}{\cosh\left(0.866\,\frac{l'}{h}\right)}$$

where

x = Horizontal distance of a point on base of tank in the direction of seismic force, from the center of tank.

COMMENTARY

C4.9.1 – Impulsive Hydrodynamic Pressure

The expressions for hydrodynamic pressure on wall and base of circular and rectangular tanks are based on work of Housner (1963a).

These expressions are for tanks with rigid walls. Wall flexibility does not affect convective pressure distribution, but can have substantial influence on impulsive pressure distribution in tall tanks. The effect of wall flexibility on impulsive pressure distribution is discussed by Veletsos (1984).

Qualitative description of impulsive pressure distribution on wall and base is given in Figure C-1b.

Vertical and horizontal distances, i.e., x and y and circumferential angle, ϕ, and strip length l' are described in Figure 8a.

PROVISIONS

(b) For Rectangular Tank (Figure 8b)

Lateral hydrodynamic impulsive pressure on wall p_{iw}, is given by

$$p_{iw} = Q_{iw}(y)\,(A_h)_i\,\rho g h$$

where, $Q_{iw}(y)$ is same as that for a circular tank and can be read from Figure 9a, with h/L being used in place of h/D.

Impulsive hydrodynamic pressure in vertical direction, on the base slab ($y = 0$), is given by:

$$p_{ib} = Q_{ib}(x)\,(A_h)_i\,\rho g h$$

$$Q_{ib}(x) = \frac{\sinh\left(1.732\,\frac{x}{h}\right)}{\cosh\left(0.866\,\frac{L}{h}\right)}$$

The value of coefficient of impulsive hydrodynamic pressure on base $Q_{ib}(x)$, can also be read from Figure 9b.

4.9.2 – Convective Hydrodynamic Pressure

The convective pressure exerted by the oscillating liquid on the tank wall and base shall be calculated as follows:

(a) Circular Tank (Figure 8a)

Lateral convective pressure on the wall p_{cw}, is given by

$$p_{cw} = Q_{cw}(y)(A_h)_c\,\rho g\,D\left[1 - \frac{1}{3}\cos^2\phi\right]\cos\phi$$

$$Q_{cw}(y) = 0.5625\,\frac{\cosh\left(3.674\,\frac{y}{D}\right)}{\cosh\left(3.674\,\frac{h}{D}\right)}$$

The value of $Q_{cw}(y)$ can also be read from Figure 10a.

Convective pressure in vertical direction, on the base slab $(y = 0)$ is given by

COMMENTARY

C4.9.2 – Convective Hydrodynamic Pressure

The expressions for hydrodynamic pressure on wall and base of circular and rectangular tanks are based on work of Housner (1963a).

Qualitative description of convective pressure distribution on wall and base is given in Figure C-1d.

PROVISIONS

$$p_{cb} = Q_{cb}(x)(A_h)_c \rho g\, D$$

where

$$Q_{cb}(x) = 1.125\left[\frac{x}{D} - \frac{4}{3}\left(\frac{x}{D}\right)^3\right]\operatorname{sech}\left(3.674\frac{h}{D}\right)$$

The value of $Q_{cb}(x)$ may also be read from Figure 10b.

(b) Rectangular Tank (Figure 8b)

The hydrodynamic pressure on the wall p_{cw}, is given by

$$p_{cw} = Q_{cw}(y)(A_h)_c \rho g\, L$$

$$Q_{cw}(y) = 0.4165\frac{\cosh\left(3.162\frac{y}{L}\right)}{\cosh\left(3.162\frac{h}{L}\right)}$$

The value of $Q_{cw}(y)$ can also be obtained from Figure 11a.

The pressure on the base slab ($y = 0$) is given by

$$p_{cb} = Q_{cb}(x)(A_h)_c \rho g\, L$$

$$Q_{cb}(x) = 1.25\left[\frac{x}{L} - \frac{4}{3}\left(\frac{x}{L}\right)^3\right]\operatorname{sech}\left(3.162\frac{h}{L}\right)$$

The value of $Q_{cb}(x)$ can also be obtained from Figure 11b.

4.9.3 –

In circular tanks, hydrodynamic pressure due to horizontal excitation varies around the circumference of the tank. However, for convenience in stress analysis of the tank wall, the hydrodynamic pressure on the tank wall may be approximated by an outward pressure distribution of intensity equal to that of the maximum hydrodynamic pressure (Figure 12a).

4.9.4 –

Hydrodynamic pressure due to horizontal excitation has curvilinear variation along wall height. However, in the absence of more

COMMENTARY

C4.9.3 –

This clause is adapted from Priestley et al. (1986). Since hydrodynamic pressure varies slowly in the circumferential direction, the design stresses can be obtained by considering pressure distribution to be uniform along the circumferential direction.

C4.9.4 –

Equivalent linear distribution of pressure along wall height is described in Figures 12b and 12c, respectively, for impulsive and convective

PROVISIONS

exact analysis, an equivalent linear pressure distribution may be assumed so as to give the same base shear and bending moment at the bottom of tank wall (Figures 12b and 12c).

4.9.5 – Pressure Due to Wall Inertia

Pressure on tank wall due to its inertia is given by

$$p_{ww} = (A_h)_i \, t \, \rho_m \, g$$

where

ρ_m = Mass density of tank wall, and

t = Wall thickness.

COMMENTARY

pressure.

For circular tanks, maximum hydrodynamic force per unit circumferential length at ϕ = 0, for impulsive and convective mode, is given by

$$q_i = \frac{(A_h)_i m_i}{\pi D/2} g \quad \text{and} \quad q_c = \frac{(A_h)_c m_c}{\pi D/2} g$$

For rectangular tanks, maximum hydrodynamic force per unit length of wall for impulsive and convective mode is given by

$$q_i = \frac{(A_h)_i m_i}{2B} g \quad \text{and} \quad q_c = \frac{(A_h)_c m_c}{2B} g$$

The equivalent linear pressure distribution for impulsive and convective modes, shown in Figure 12b and 12c can be obtained as:

$$a_i = \frac{q_i}{h^2}(4h - 6h_i) \text{ and } b_i = \frac{q_i}{h^2}(6h_i - 2h)$$

$$a_c = \frac{q_c}{h^2}(4h - 6h_c) \text{ and } b_c = \frac{q_c}{h^2}(6h_c - 2h)$$

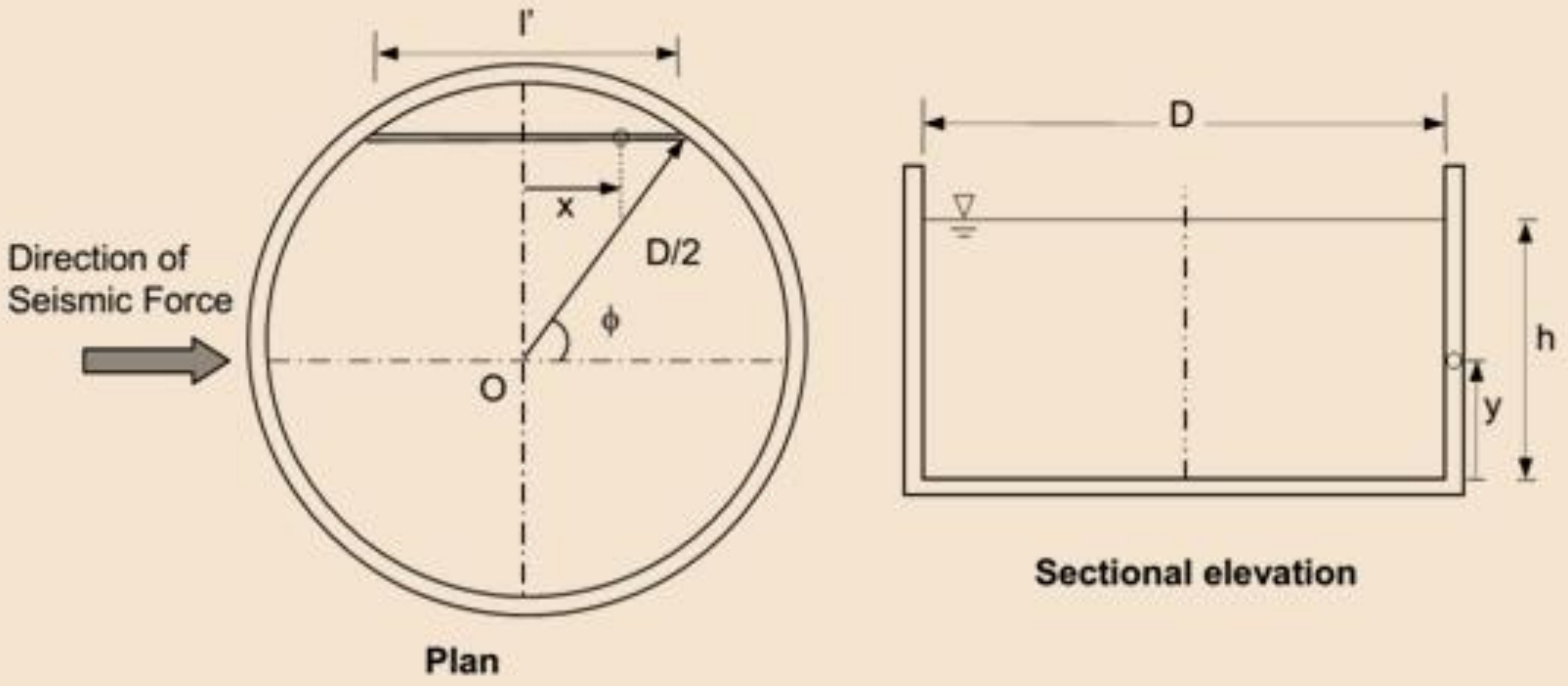
C4.9.5 – Pressure Due to Wall Inertia

Pressure due to wall inertia will act in the same direction as that of seismic force.

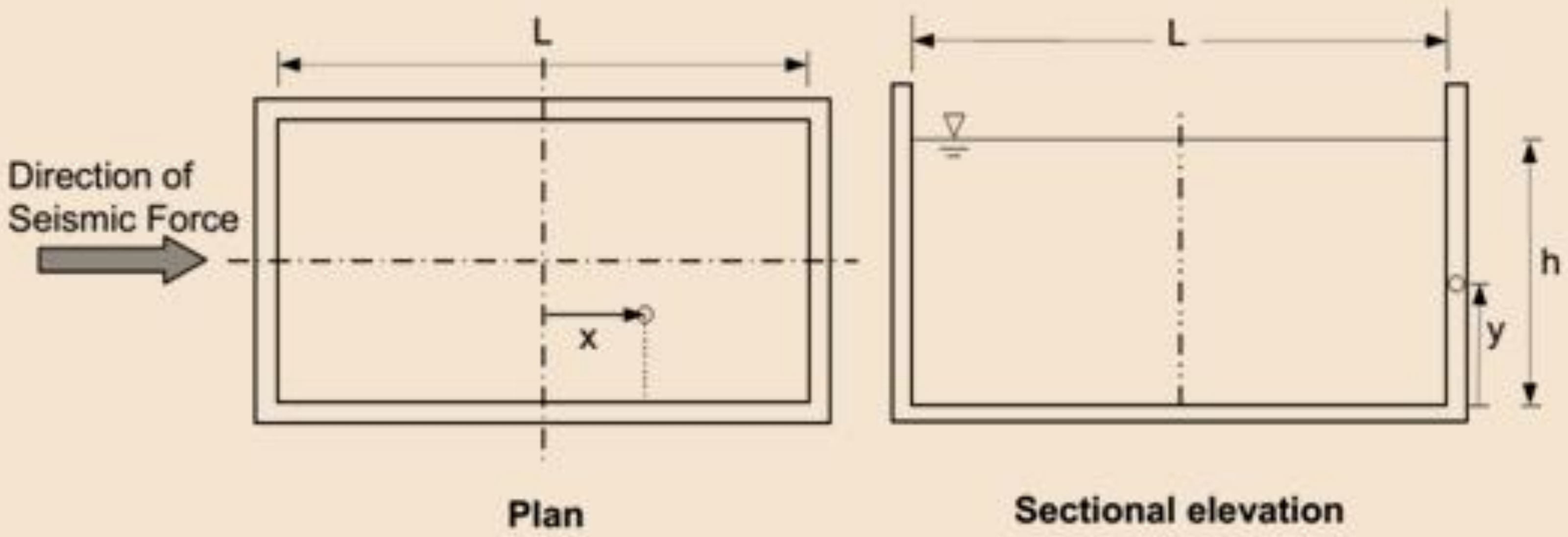
For steel tanks, wall inertia may not be significant. However, for concrete tanks, wall inertia may be substantial.

Pressure due to wall inertia, which is constant along the wall height for walls of uniform thickness, should be added to impulsive hydrodynamic pressure.

PROVISIONS



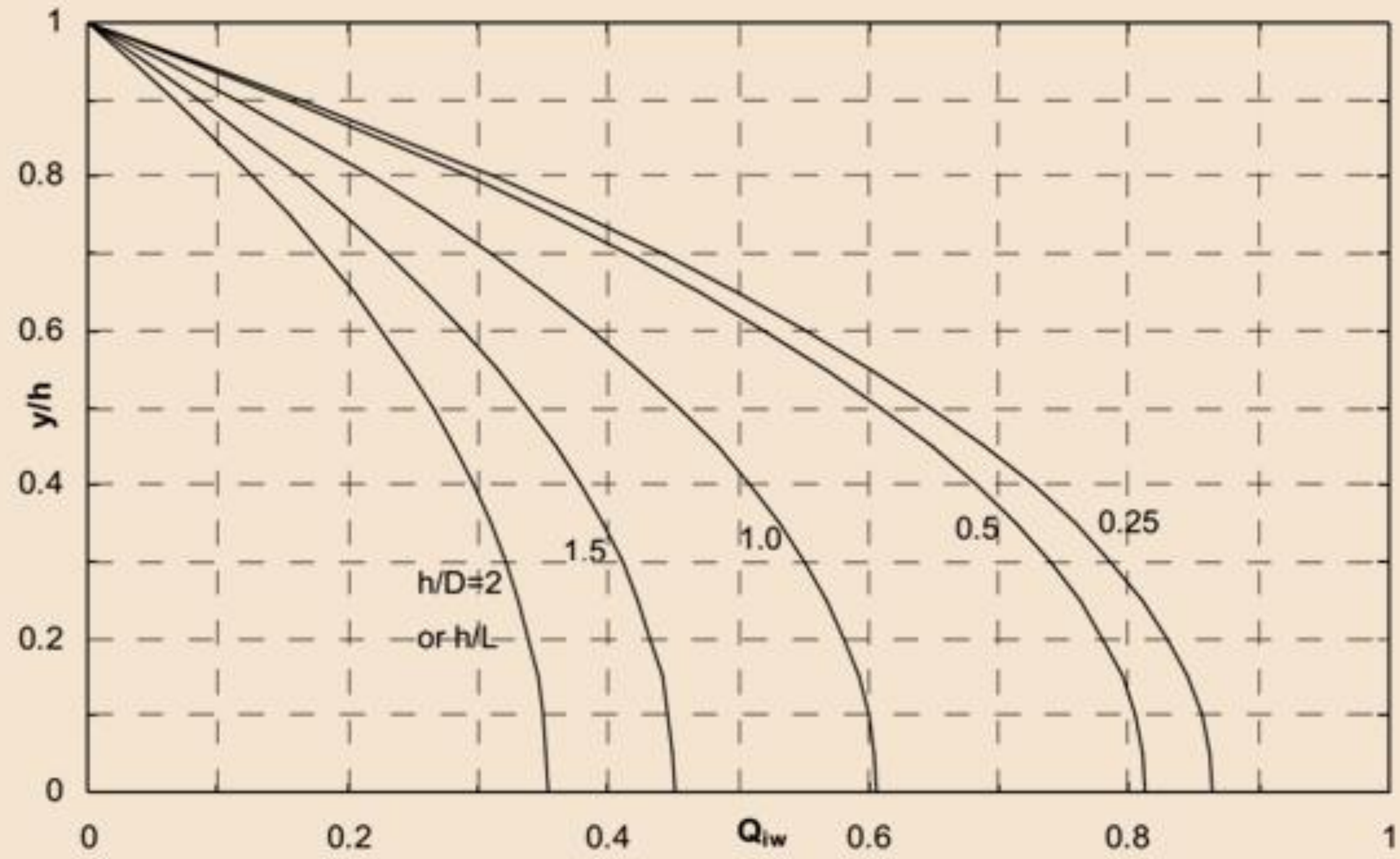
(a) Circular tank



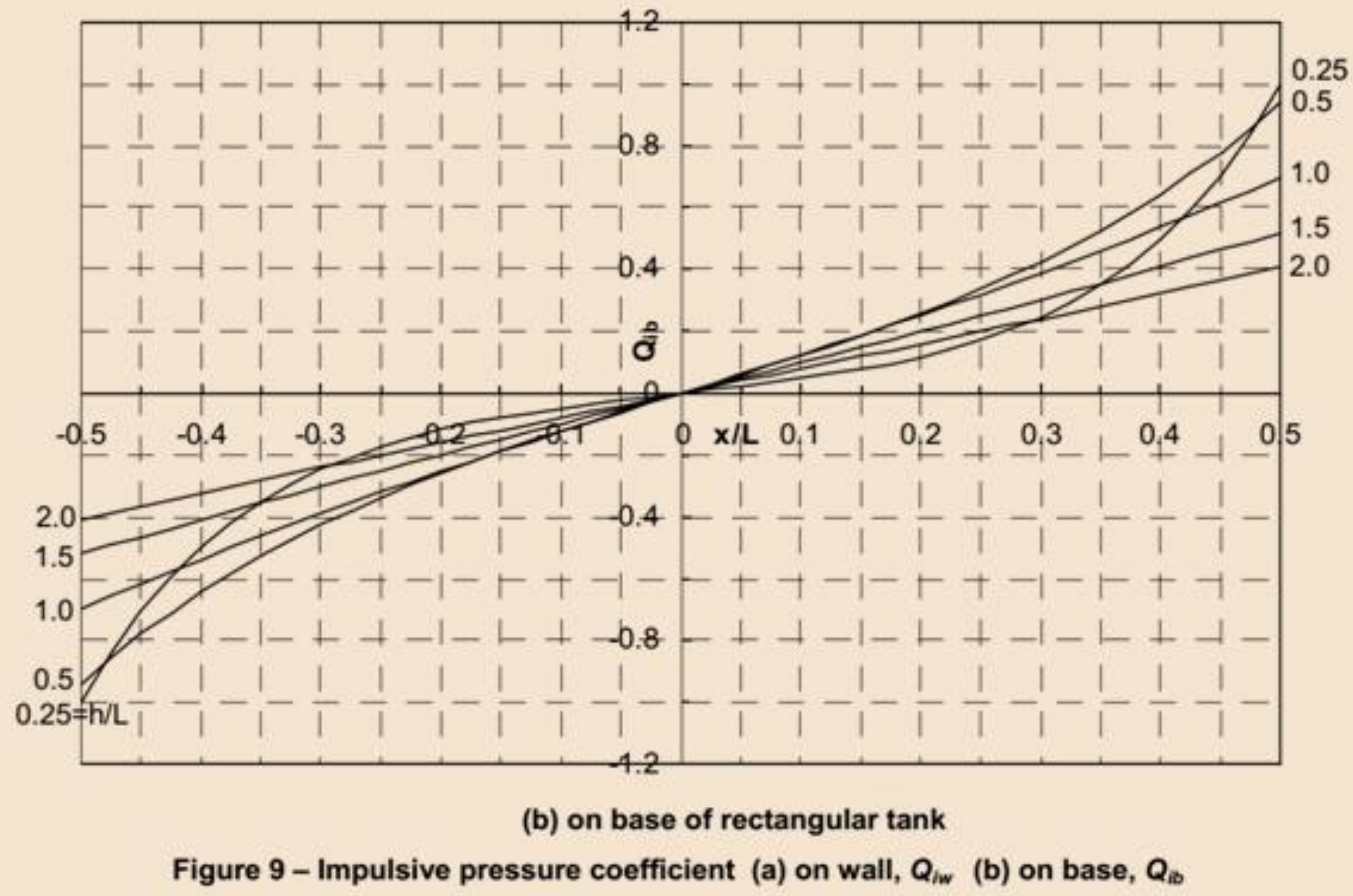
(b) Rectangular tank

Figure 8 – Geometry of (a) Circular tank and (b) Rectangular tank

PROVISIONS



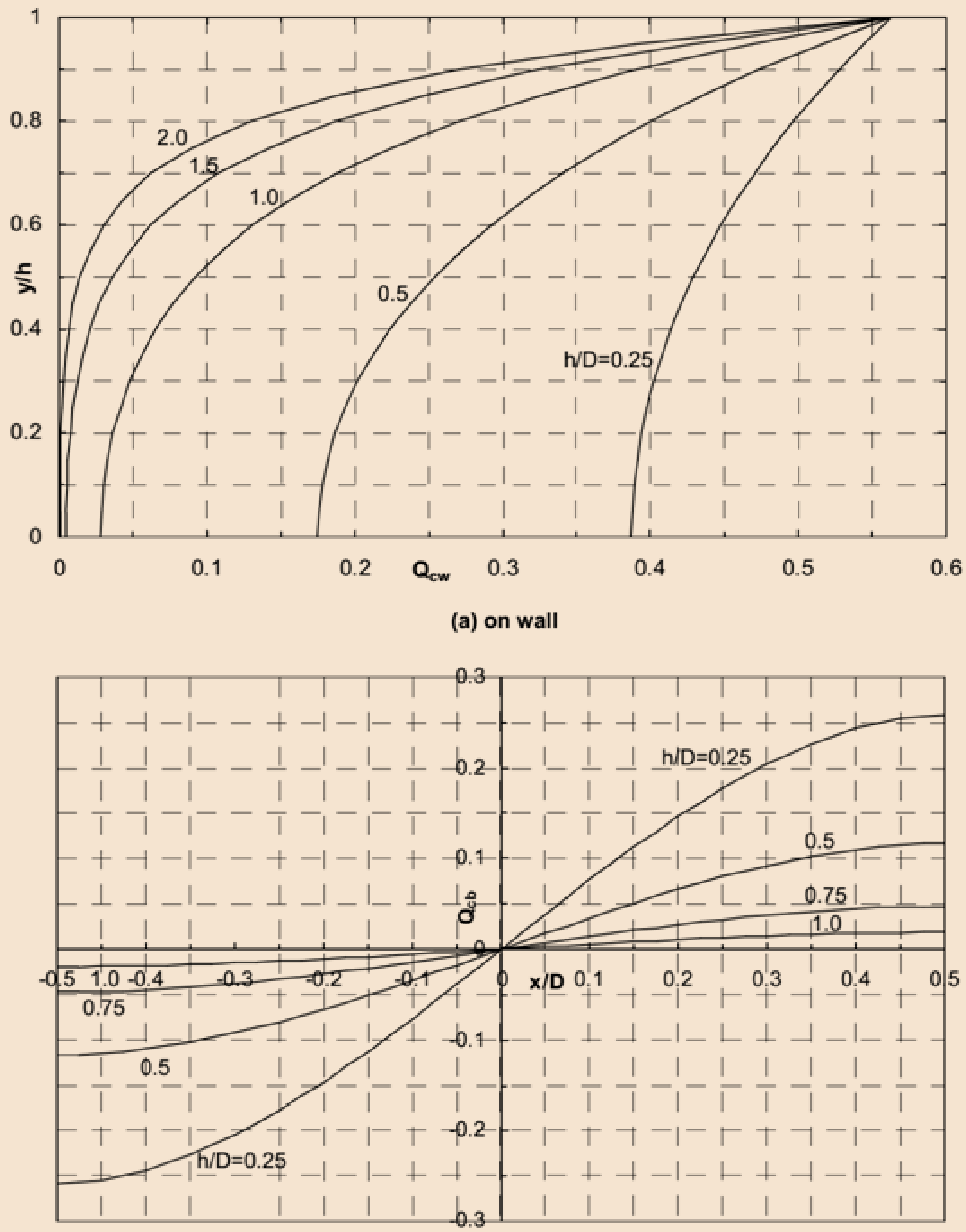
(a) on wall of circular and rectangular tank



(b) on base of rectangular tank

Figure 9 – Impulsive pressure coefficient (a) on wall, Q_{iw} (b) on base, Q_{ib}

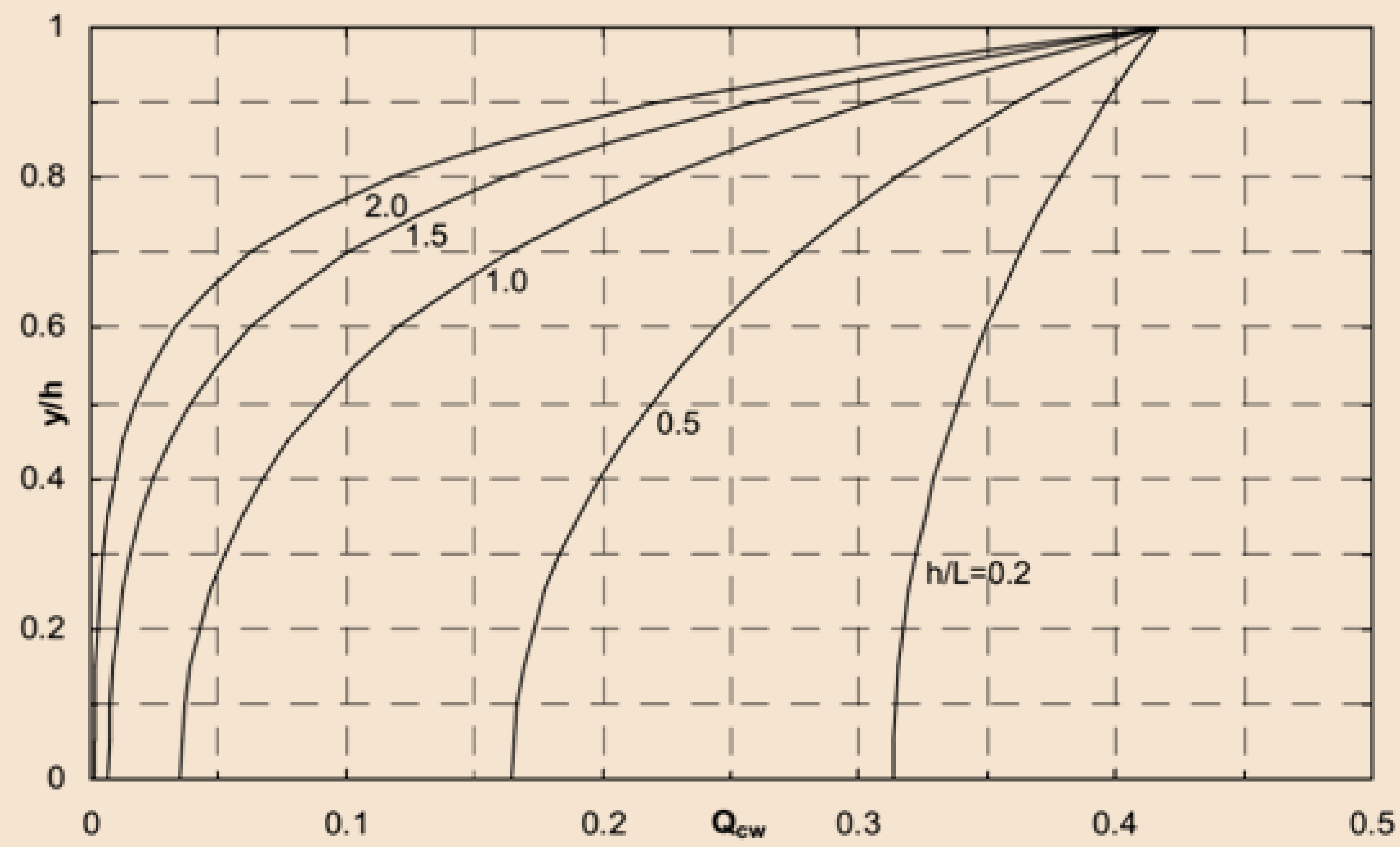
PROVISIONS



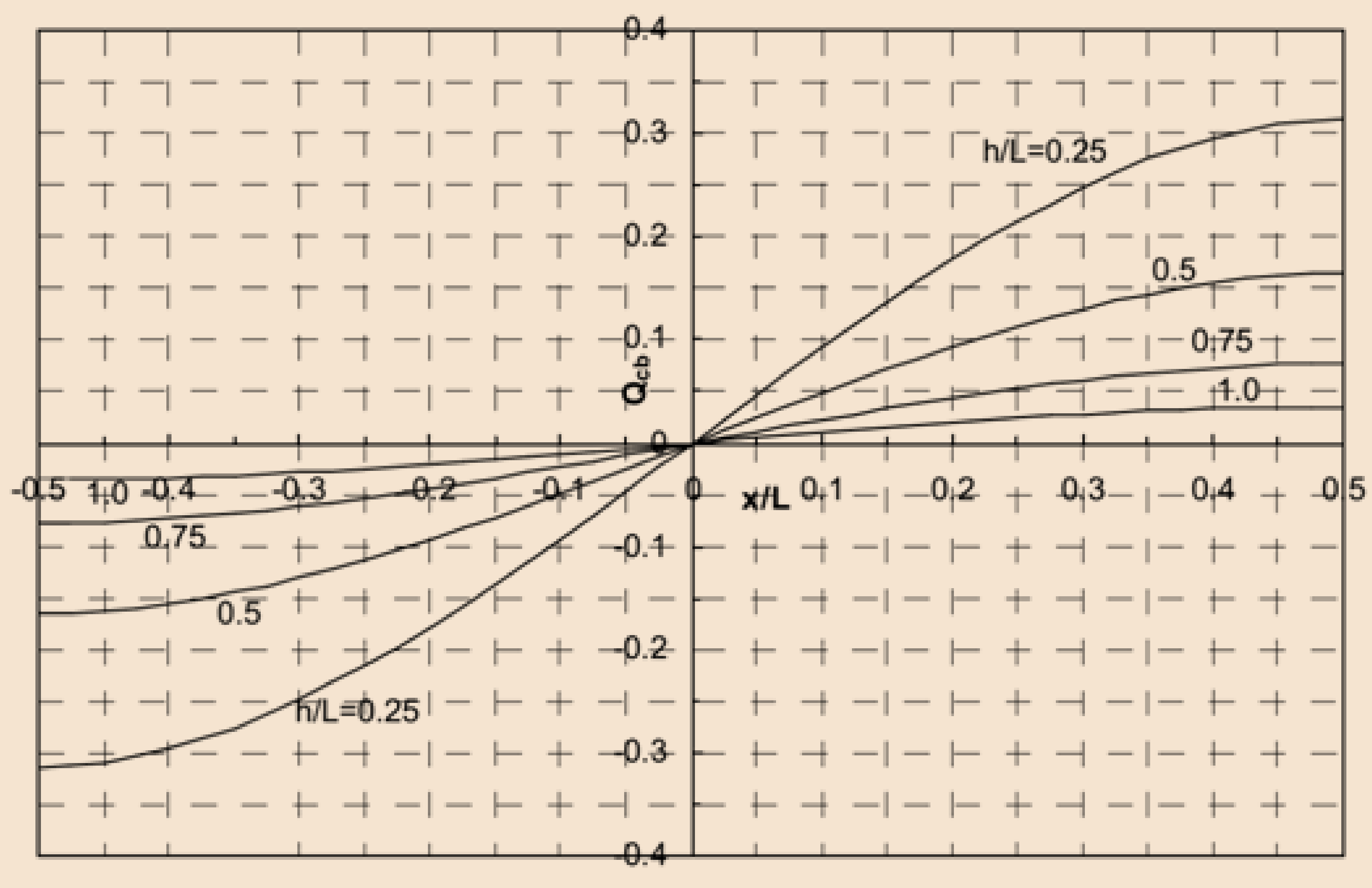
(b) on base

Figure 10 Convective pressure coefficient for circular tank (a) on wall, Q_{cw} (b) on base, Q_{cb}

PROVISIONS



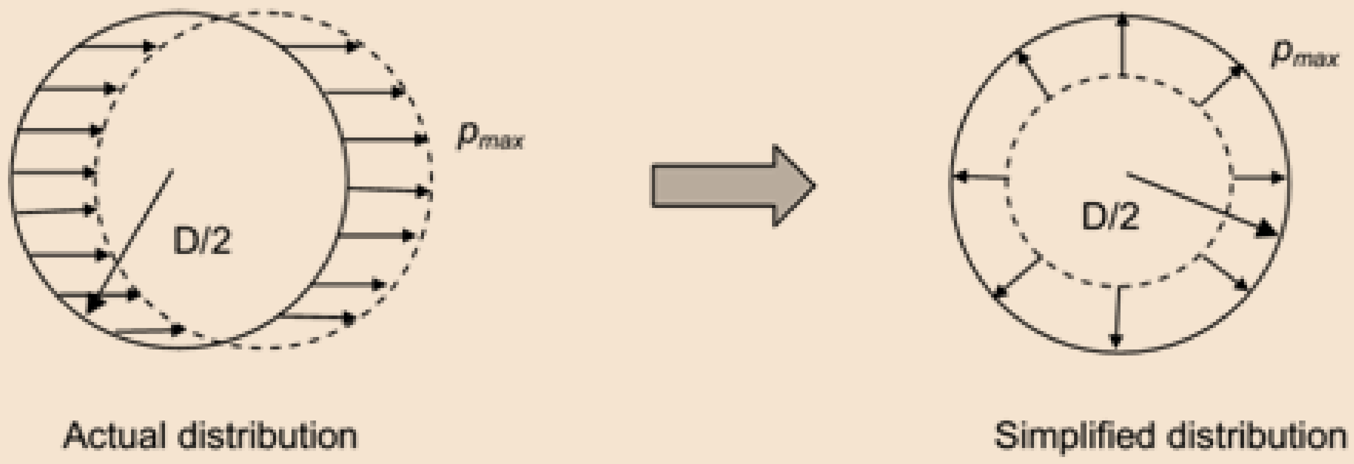
(a) on wall



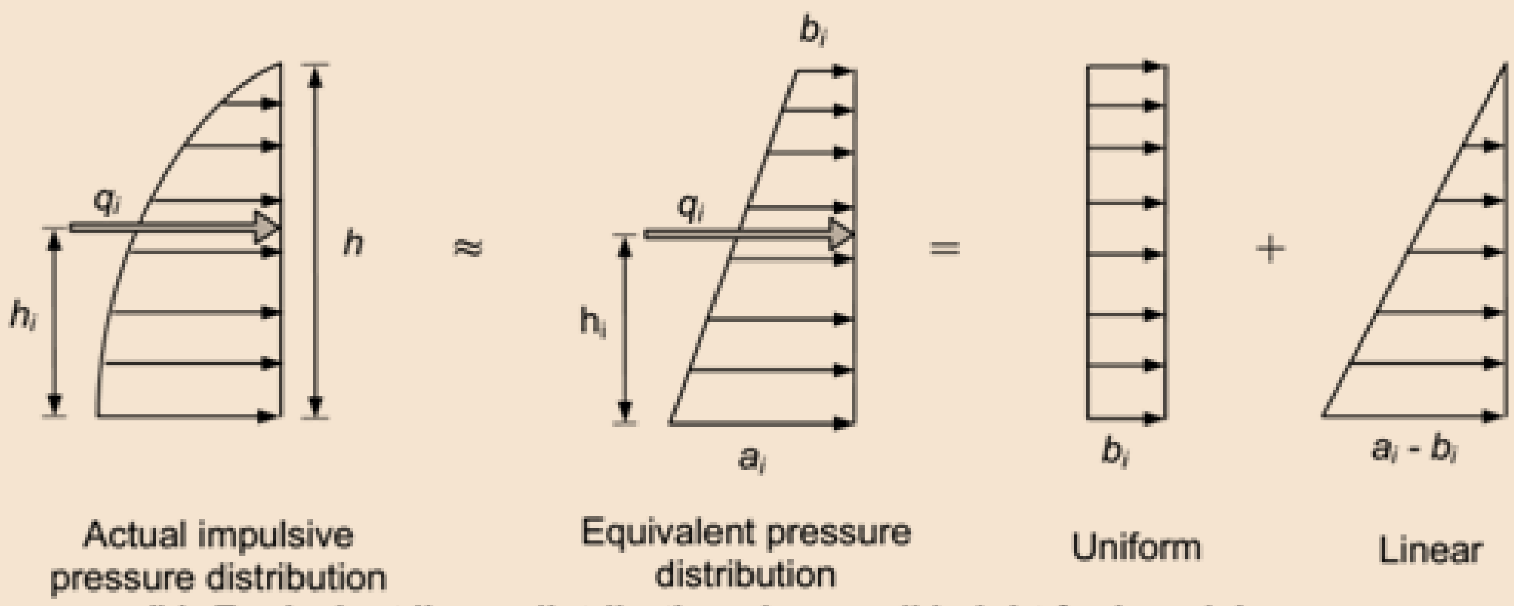
(b) on base

Figure 11 Convective pressure coefficient for rectangular tank (a) on wall, Q_{cw} (b) on base, Q_{cb}

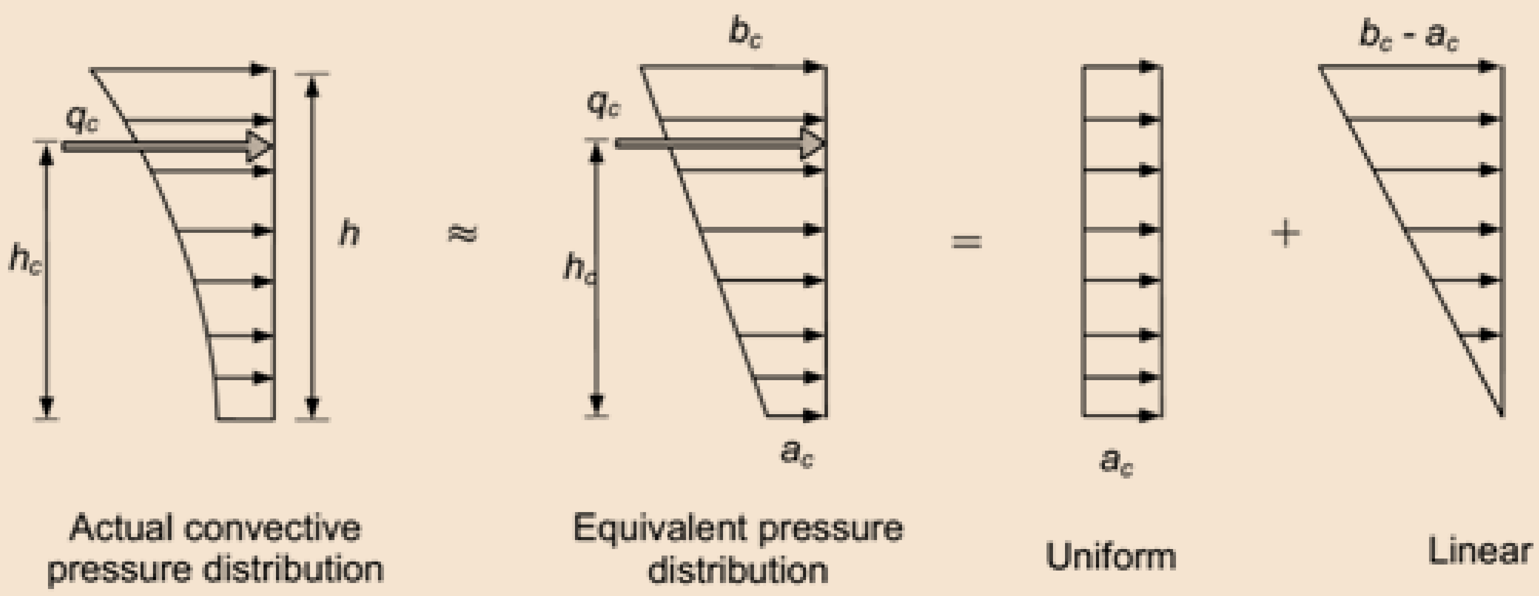
PROVISIONS



(a) Simplified pressure distribution in circumferential direction on tank wall



(b) Equivalent linear distribution along wall height for impulsive pressure



(c) Equivalent linear distribution along wall height for convective pressure

Figure 12 – Hydrodynamic pressure distribution for wall analysis

PROVISIONS

4.10 – Effect of Vertical Ground Acceleration

Due to vertical ground acceleration, effective weight of liquid increases, this induces additional pressure on tank wall, whose distribution is similar to that of hydrostatic pressure.

4.10.1 –

Hydrodynamic pressure on tank wall due to vertical ground acceleration may be taken as

$$p_v = (A_v)\rho g h(1 - y/h)$$

$$A_v = \frac{2}{3}\left(\frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}\right)$$

where

y = vertical distance of point under consideration from bottom of tank wall, and

$\frac{S_a}{g}$ = Average response acceleration coefficient given by Figure 2 and Table 3 of IS 1893 (Part 1):2002 and subject to Clauses 4.5.2 and 4.5.3 of this code.

In absence of more refined analysis, time period of vertical mode of vibration for all types of tank may be taken as 0.3 sec.

4.10.2 –

The maximum value of hydrodynamic pressure should be obtained by combining pressure due to horizontal and vertical excitation through square root of sum of squares (SRSS) rule, which can be given as

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}^2 + p_v^{\ 2}}$$

COMMENTARY

C4.10 – Effect of Vertical Ground Acceleration

Vertical ground acceleration induces hydrodynamic pressure on wall in addition to that due to horizontal ground acceleration. In circular tanks, this pressure is uniformly distributed in the circumferential direction.

C4.10.1 –

Distribution of hydrodynamic pressure due to vertical ground acceleration is similar to that of hydrostatic pressure. This expression is based on rigid wall assumption. Effect of wall flexibility on hydrodynamic pressure distribution is described in Eurocode 8 (1998).

Design vertical acceleration spectrum is taken as two-third of design horizontal acceleration spectrum, as per clause 6.4.5 of IS 1893 (Part1).

To avoid complexities associated with the evaluation of time period of vertical mode, time period of vertical mode is assumed as 0.3 seconds for all types of tanks. However, for ground supported circular tanks, expression for time period of vertical mode of vibration (i.e., breathing mode) can be obtained using expressions given in ACI 350.3 (2001) and Eurocode 8 (1998).

While considering the vertical acceleration, effect of increase in weight density of tank and its content may also be considered.

PROVISIONS

4.11 – Sloshing Wave Height

Maximum sloshing wave height is given by

$$d_{max} = (A_h)_c\, R\, \frac{D}{2} \qquad \text{For circular tank}$$

$$d_{max} = (A_h)_c\, R\, \frac{L}{2} \qquad \text{For rectangular tank}$$

where

$(A_h)_c$ = Design horizontal seismic coefficient corresponding to convective time period.

4.12 – Anchorage Requirement

Circular ground supported tanks shall be anchored to their foundation (Figure 13) when

$$\frac{h}{D} > \frac{1}{(A_h)_i}$$

In case of rectangular tank, the same expression may be used with *L* instead of *D*.

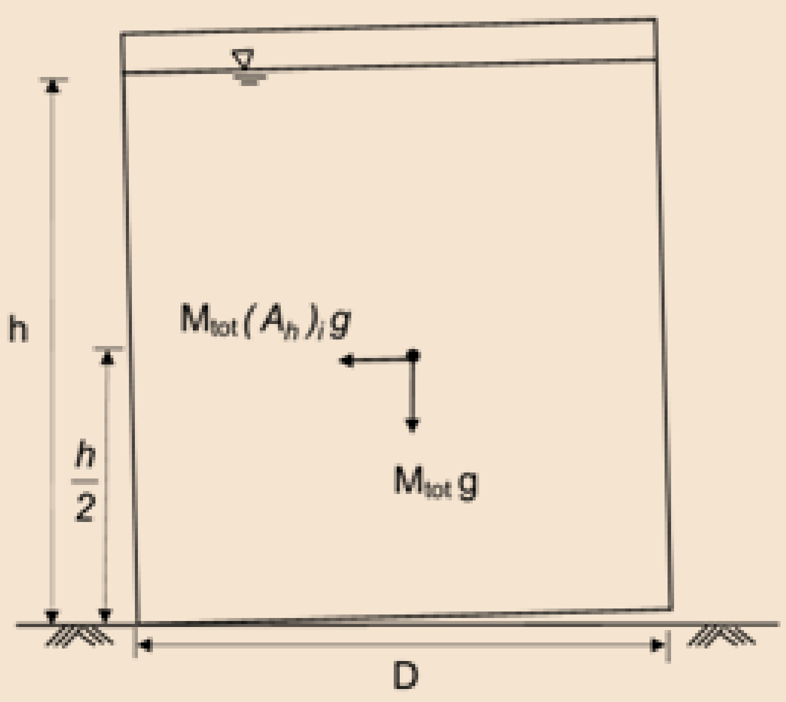


Figure 13 – Initiation of rocking of tank

COMMENTARY

C4.11 – Sloshing Wave Height

Expression for maximum sloshing wave height is taken from ACI 350.3 (2001).

Free board to be provided in a tank may be based on maximum value of sloshing wave height. This is particularly important for tanks containing toxic liquids, where loss of liquid needs to be prevented. If sufficient free board is not provided roof structure should be designed to resist the uplift pressure due to sloshing of liquid.

Moreover, if there is obstruction to free movement of convective mass due to insufficient free board, the amount of liquid in convective mode will also get changed. More information regarding loads on roof structure and revised convective mass can be obtained in Malhotra (2004).

C4.12 – Anchorage Requirement

This condition is described by Priestley et al. (1986). Consider a tank which is about to rock (Figure 13). Let M_{tot} denotes the total mass of the tank-liquid system, D denote the tank diameter, and $(A_h)_i\, g$ denote the peak response acceleration. Taking moments about the edge,

$$M_{tot}(A_h)_i\, g\, \frac{h}{2} = M_{tot}\, g\, \frac{D}{2}$$

$$\frac{h}{D} = \frac{1}{(A_h)_i}$$

Thus, when h/D exceeds the value indicated above, the tank should be anchored to its foundation. The derivation assumes that the entire liquid responds in the impulsive mode. This approximation is reasonable for tanks with high h/D ratios that are susceptible to overturning.

PROVISIONS

4.13 – Miscellaneous

4.13.1 — Piping

Piping systems connected to tanks shall consider the potential movement of the connection points during earthquake and provide for sufficient flexibility to avoid damage. The piping system shall be designed so as not to impart significant mechanical loading on tank. Local loads at pipe connections can be considered in the design of the tank. Mechanical devices, which add flexibility to piping such as bellows, expansion joints and other special couplings, may be used in the connections.

4.13.2 – Buckling of Shell

Ground supported tanks (particularly, steel tanks) shall be checked for failure against buckling. Similarly, safety of shaft type of staging of elevated tanks against buckling shall be ensured.

4.13.3 – Buried Tanks

Dynamic earth pressure shall be taken into account while computing the base shear of a partially or fully buried tank. Earth pressure shall also be considered in the design of walls. In buried tanks, dynamic earth pressure shall not be relied upon to reduce dynamic effects due to liquid.

COMMENTARY

C4.13 – Miscellaneous

C4.13.1 – Piping

FEMA 368 (2000) provides more information on flexibility requirements of piping system.

C4.13.2 – Buckling of Shell

More information of buckling of steel tanks is given by Priestley et al. (1986).

C4.13.3 – Buried Tanks

The value of response reduction factor for buried tanks is given in Table 2.

For buried tanks, the analysis procedure remains same as that for ground supported tank except for consideration of dynamic earth pressure. For effect of dynamic earth pressure, following comments from Munshi and Sherman (2004) are taken:

The effect of dynamic earth pressure is commonly approximated by Monobe-Okabe theory (1992). This involves the use of constant horizontal and vertical acceleration from the earthquake acting on the soil mass comprising Coulomb's active or passive wedge. This theory assumes that wall movements are sufficient to fully mobilize the shear resistance along the backfill wedge. In sufficiently rigid tanks (such as concrete tanks), the wall deformation and consequent movement into the surrounding soil is usually small enough that the active or passive soil wedge is not fully activated. For dense, medium-dense, and loose sands, a deformation equal to 0.1, 0.2, and 0.4%, respectively, of wall height is necessary to activate the active soil reaction (Ebeling, R.M. and Morrison, E.E. (1993) and Clough, G. W.

PROVISIONS

COMMENTARY

and Duncan, J.M. (1991)). Similarly, a deformation of 1, 2, and 4% of the wall height is required to activate the passive resistance of these sands. Therefore, determination of dynamic active and passive pressures may not be necessary when wall deformations are small. Dynamic earth pressure at rest should be included, however, as given by the following equation by Clough and Duncan (1991)

$$F = k_h \gamma_s H_s^2$$

where k_h is the dynamic coefficient of earth pressure; γ_s is the density of the soil; and H_s is the height of soil being retained. This force acting at height $0.6h$ above the base should be used to increase or decrease the at-rest pressure when wall deformations are small.

4.13.4 – Shear Transfer

The lateral earthquake force generates shear between wall and base slab and between roof and wall. Wall-to-base slab, wall-to-roof slab and wall-to-wall joints shall be suitably designed to transfer shear forces. Similarly in elevated tanks, connection between container and staging should be suitably designed to transfer the shear force.

4.13.5 – P- Delta Effect

For elevated tanks with tall staging (say, staging height more than five times the least lateral dimension) it may be required to include the P-Delta effect. For such tall tanks, it must also be confirmed that higher modes of staging do not have significant contribution to dynamic response.

C4.13.5 – P-Delta Effect

P-delta effect could be significant in elevated tanks with tall staging. P-delta effect can be minimized by restricting total lateral deflection of staging to $h_s/500$, where h_s is height of staging.

For small capacity tanks with tall staging, weight of staging can be considerable compared to total weight of tank. Hence, contribution from higher modes of staging shall also be ascertained. If mass excited in higher modes of staging is significant then these shall be included in the analysis, and response spectrum analysis shall be performed.

IITK-GSDMA GUIDELINES for SEISMIC DESIGN of LIQUID STORAGE TANKS

Provisions with Commentary and Explanatory Examples

PART 2: EXPLANATORY EXAMPLES

Example 1 – Elevated Tank Supported on 4 Column RC Staging

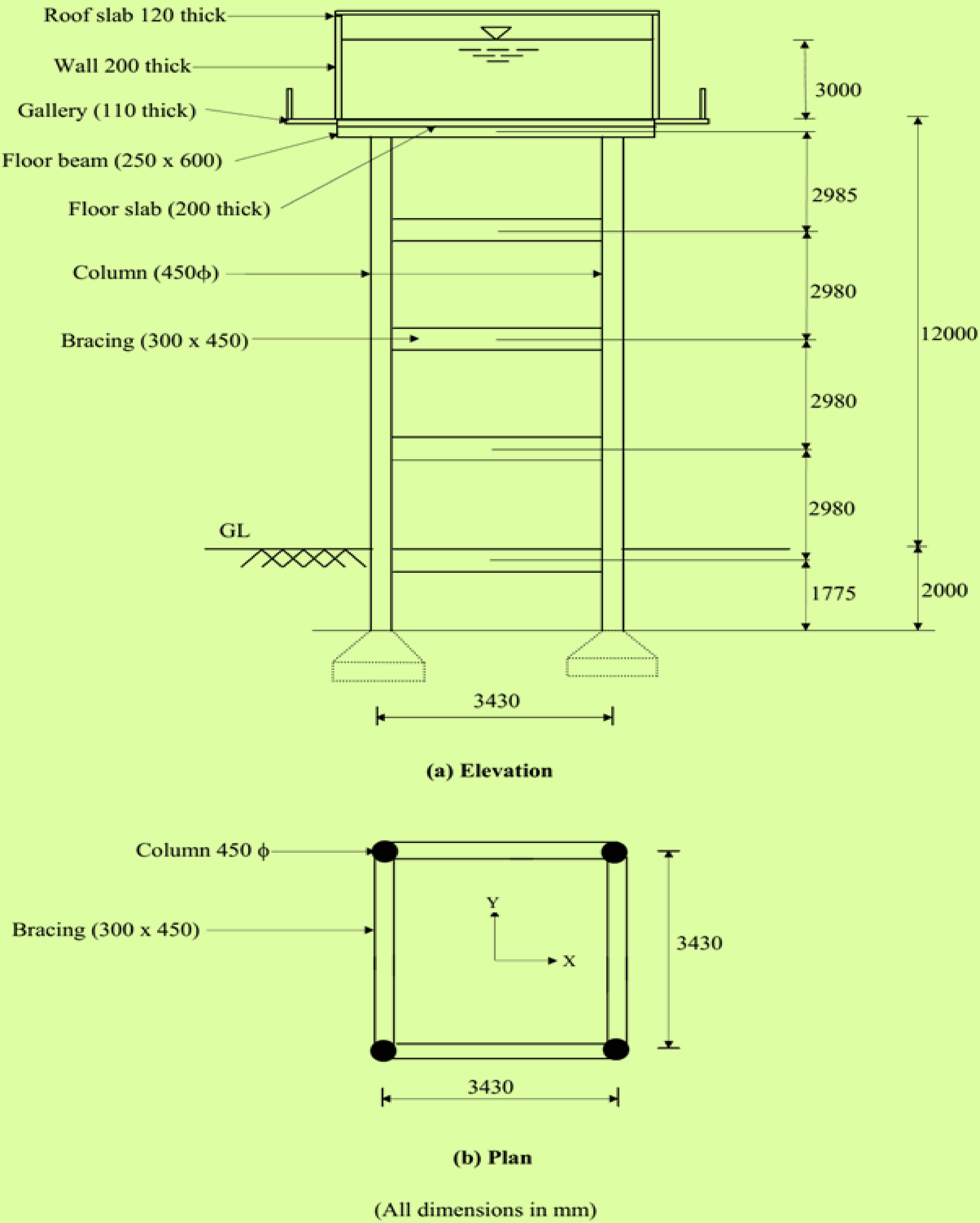


Figure 1.1 Details of tank geometry

Example 1 – Elevated Tank Supported on 4 Column RC Staging

1. Problem Statement:

A RC circular water container of 50 m^3 capacity has internal diameter of 4.65 m and height of 3.3 m (including freeboard of 0.3 m). It is supported on RC staging consisting of 4 columns of 450 mm dia with horizontal bracings of 300 x 450 mm at four levels. The lowest supply level is 12 m above ground level. Staging conforms to ductile detailing as per IS13920. Staging columns have isolated rectangular footings at a depth of 2m from ground level. Tank is located on soft soil in seismic zone II. Grade of staging concrete and steel are M20 and Fe415, respectively. Density of concrete is 25 kN/m^3. Analyze the tank for seismic loads.

Solution:

Tank must be analysed for tank full and empty conditions.

1.1. Preliminary Data

Details of sizes of various components and geometry are shown in Table 1.1 and Figure 1.1.

Table 1.1 Sizes of various components

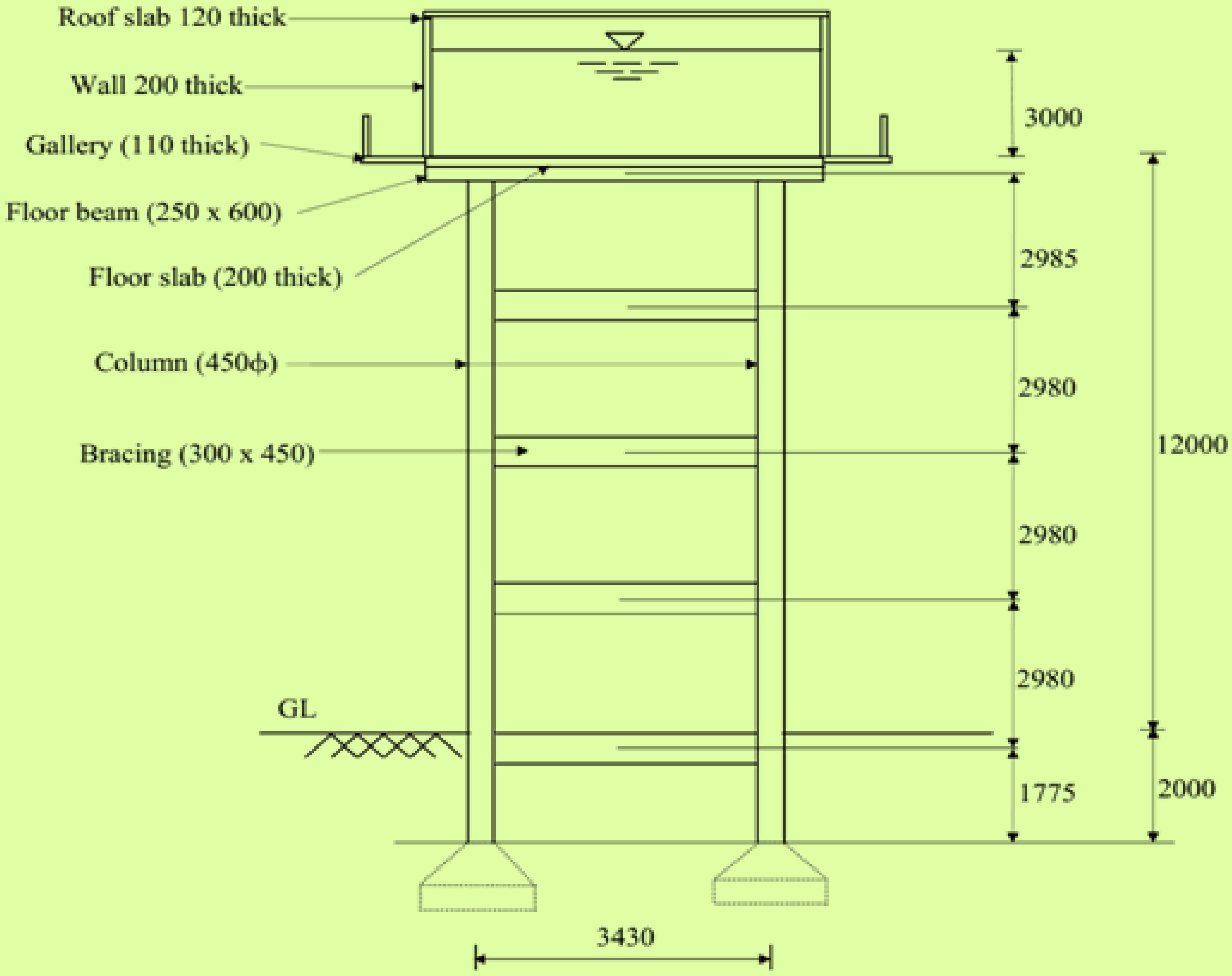
Component	Size (mm)
Roof Slab	120 thick
Wall	200 thick
Floor Slab	200 thick
Gallery	110 thick
Floor Beams	250 x 600
Braces	300 x 450
Columns	450 dia.

1.2. Weight Calculations

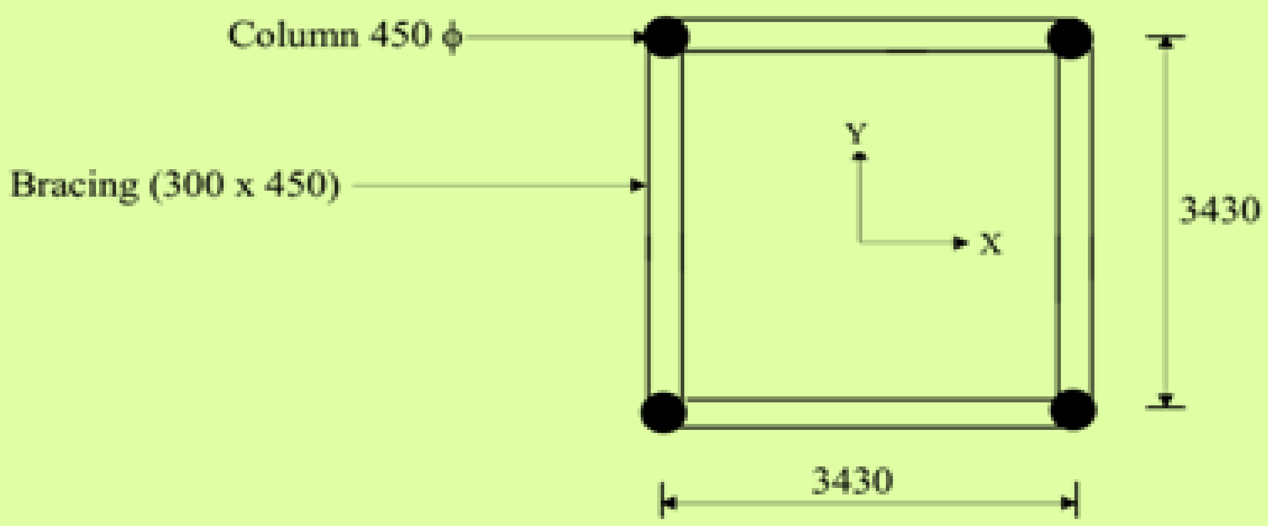
Table 1.2 Weight of various components

Component	Calculations	Weight (kN)
Roof Slab	$[\pi \times (5.05)^2 \times (0.12 \times 25)] / 4$	60.1
Wall	$\pi \times 4.85 \times 0.20 \times 3.30 \times 25$	251.4
Floor Slab	$[\pi \times (5.05)^2 \times 0.20 \times 25] / 4$	100.2
Floor Beam	$\pi \times 4.85 \times 0.25 \times (0.60 - 0.20) \times 25$	38.1
Gallery	$[\pi \times ((7.05)^2 - (5.05)^2) \times (0.110 \times 25)] / 4$	52.3
Columns	$[\pi \times (0.45)^2 \times 11.7 \times 4 \times 25] / 4$	186.1
Braces	$3.43 \times 0.30 \times 0.45 \times 4 \times 4 \times 25$	185.2
Water	$[\pi \times 4.65^2 \times 3.0 \times 9.81] / 4$	499.8

Note: i) Weights of floor finish and plaster should be accounted, wherever applicable.
ii) Live load on roof slab and gallery is not considered for seismic load computations.
iii) Water load is considered as dead load.
iv) For seismic analysis, freeboard is not included in depth of water.



(a) Elevation



(b) Plan

(All dimensions in mm)

Figure 1.1 Details of tank geometry

From Table 1.2,

Weight of staging = 186.1 + 185.2 = 371.3 kN.

Weight of empty container = 60.1 + 251.4 + 100.2 + 38.1 + 52.3 = 502.1 kN.

Hence, weight of container + one third weight of staging = 502.1 + 371.3 / 3 = 626 kN.

1.3. Center of Gravity of Empty Container

Components of empty container are: roof slab, wall, floor slab, gallery and floor beam. With reference to Figure 1.2, height of CG of empty container from top of floor slab will be

$$= [(60.1 \times 3.36) + (251.4 \times 1.65) - (100.2 \times 0.1) - (52.3 \times 0.055) - (38.1 \times 0.4)] / 502.1$$

$$= 1.18 \text{ m.}$$

Hence, height of CG of empty container from top of footing will be 14 + 1.18 = 15.18 m.

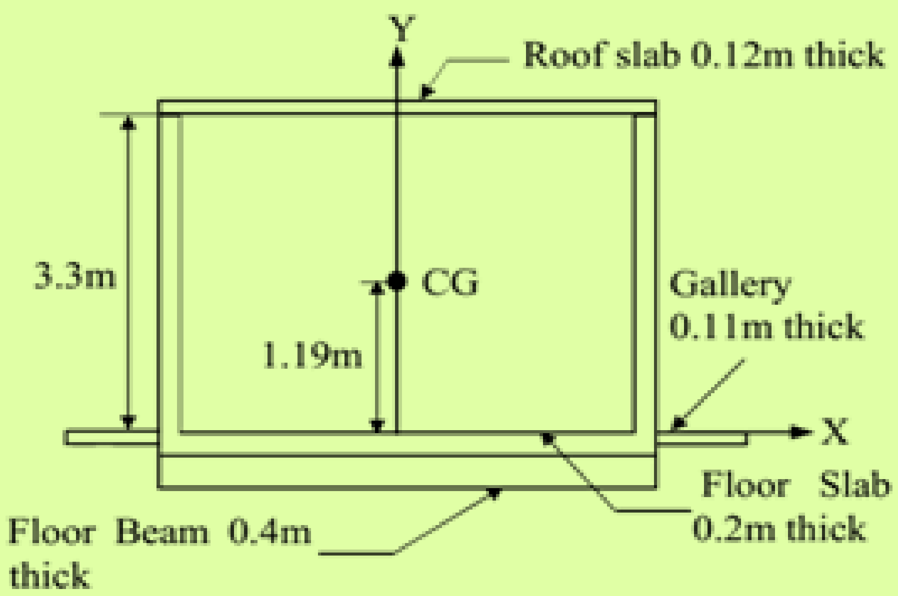


Figure 1.2 CG of empty container

1.4. Parameters of Spring Mass Model

Weight of water = 499.8 kN = 4,99,800 N.

Hence, mass of water, m = 4,99,800 / 9.81 = 50,948 kg.

Depth of water, h = 3.0 m.

Inner diameter of the tank, D = 4.65m.

Hence, for h / D = 3.0 / 4.65 = 0.65,

$m_i / m = 0.65$; $m_i = 0.65 \times 50{,}948 = 33{,}116$ kg.

$m_c / m = 0.35$; $m_c = 0.35 \times 50{,}948 = 17{,}832$ kg

$h_i / h = 0.375$; $h_i = 0.375 \times 3.0 = 1.13$ m

$h_i^* / h = 0.64$; $h_i^* = 0.64 \times 3.0 = 1.92$ m

$h_c / h = 0.65$; $h_c = 0.65 \times 3.0 = 1.95$

$h_c^* / h = 0.73$; $h_c^* = 0.73 \times 3.0 = 2.19$ m.

(Section 4.2.2.2)

Note that the sum of impulsive and convective masses is 50,948 kg which compares well with the total mass. However in some cases, there may be difference of 2 to 3%.

Mass of empty container + one third mass of staging

$m_s = (502.1 + 371.3 / 3) \times (1{,}000 / 9.81)$

$= 63{,}799$ kg.

1.5. Lateral Stiffness of Staging

Lateral stiffness of staging is defined as the force required to be applied at the CG of tank so as to get a corresponding unit deflection. As per Section 4.3.1.3, CG of tank is the combined CG of empty container and impulsive mass. However, in this example, CG of tank is taken as CG of empty container.

From the deflection of CG of tank due to an arbitrary lateral force one can get the stiffness of staging.

Finite element software is used to model the staging (Refer Figure 1.3). Modulus of elasticity for M20 concrete is obtained as $5{,}000\sqrt{f_{ck}}$ = $5{,}000 \times \sqrt{20}$ = 22,360 MPa or 22.36 x 10^6 kN/m^2. Since container portion is quite rigid, a rigid link is assumed from top of staging to the CG of tank. In FE model of staging, length of rigid link is =1.18 + 0.3 = 1.48 m.

Further, to account for the rigidity imparted due to floor slab, floor beams are modeled as T-beams. Here, stiffness of staging is to be obtained in X-direction (Refer Figure 1.1), hence, one single frame of staging can be analysed in this case.

From the analysis, deflection of CG of tank due to an arbitrary 10 kN force is obtained as 0.00330 m.

Thus, lateral stiffness of one frame of staging

= 10 /0.00330 = 3,030 kN/m.

Since staging consists of two such frames, total lateral stiffness of staging,

$K_s = 2 \times 3{,}030 = 6{,}060$ kN/m.

Above analysis can also be performed manually by using standard structural analysis methods.

Here, analysis of staging is being performed for earthquake loading in X-direction. However, for some staging members this may not be the critical direction.

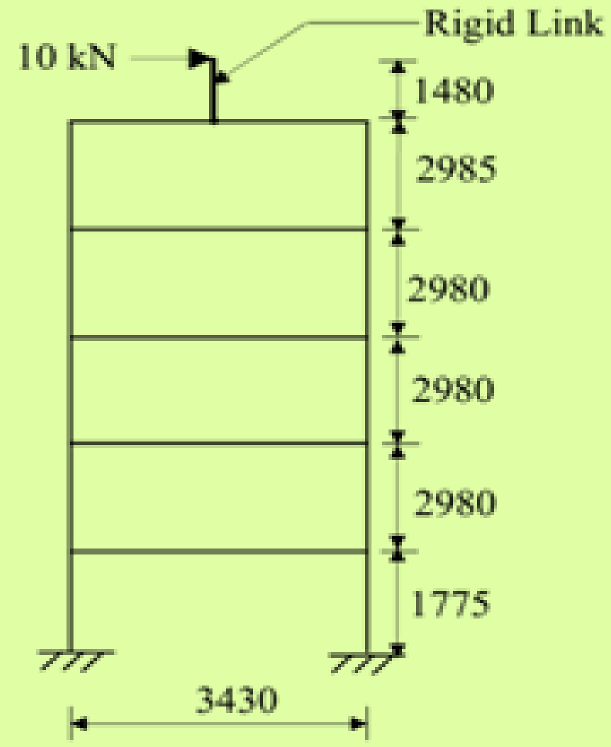


Figure 1.3 FE model of staging

1.6. Time Period

Time period of impulsive mode,

$$T_i = 2\pi\sqrt{\frac{m_i + m_s}{K_s}} \quad \text{(Section 4.3.1.3)}$$

$$= 2\pi\sqrt{\frac{33{,}116 + 63{,}799}{60{,}60{,}000}} = 0.80\,sec.$$

Time period of convective mode,

$$T_c = C_c\sqrt{\frac{D}{g}}$$

For $h / D = 0.65$, $C_c = 3.28$.

(Section 4.3.2.2 (a))

Thus, $T_c = 3.28\sqrt{\frac{4.65}{9.81}} = 2.26$ sec.

1.7. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.1$ (IS 1893(Part 1): Table 2; Zone II)

$I = 1.5$ (Table 1)

Since staging has special moment resisting frames (SMRF), R is taken as 2.5

(Table 2)

Here, $T_i = 0.80$ sec,

Site has soft soil,

Damping = 5%, (Section 4.4)

Hence, $(S_a/g)_i = 2.09$

(IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.1}{2} \times \frac{1.5}{2.5} \times 2.09 = 0.06.$$

Design horizontal seismic coefficient for convective mode,

$$(A_h)_c = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_c$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.1$ (IS 1893(Part 1): Table 2; Zone II)

$I = 1.5$ (Table 1)

$R = 2.5$

For convective mode, value of R is taken same as that for impulsive mode as per Section 4.5.1.

Here, $T_c = 2.26$ sec,

Site has soft soil,

Damping = 0.5%, (Section 4.4)

Hence, $(S_a/g)_c = 1.75 \times 0.74 = 1.3$

(IS 1893(Part 1): Figure 2)

Multiplying factor of 1.75 is used to obtain S_a/g values for 0.5% damping from that for 5% damping.

(Section 4.5.4)

$$(A_h)_c = \frac{0.1}{2} \times \frac{1.5}{2.5} \times 1.3 = 0.04$$

1.8. Base Shear

Base shear at the bottom of staging, in impulsive mode,

$$V_i = (A_h)_i\,(m_i + m_s)\,g$$

(Section 4.6.2)

$= 0.06 \times (33{,}116 + 63{,}799) \times 9.81$

$= 59.9$ kN.

Similarly, base shear in convective mode,

$V_c = (A_h)_c\, m_c\, g$ (Section 4.6.2)

$= 0.04 \times 17{,}832 \times 9.81$

$= 7.0$ kN.

Total base shear at the bottom of staging,

$V = \sqrt{V_i^2 + V_c^2}$ (Section 4.6.3)

$= \sqrt{(59.9)^2 + (7.0)^2}$

$= 60$ kN.

Total lateral base shear is about 5 % of total seismic weight (1,126 kN). It may be noted that this tank is located in seismic zone II.

1.9. Base Moment

Overturning moment at the base of staging, in impulsive mode,

$M_i^* = (A_h)_i [\, m_i\, (h_i^* + h_s) + m_s\, h_{cg}\,]\, g$

(Section 4.7.2)

$= 0.06 \times [33{,}116 \times (1.92 + 14) + (63{,}799 \times 15.18)] \times 9.81$

$= 924$ kN-m.

Similarly, overturning moment in convective mode,

$M_c^* = (A_h)_c\, m_c\, (h_c^* + h_s)\, g$ (Section 4.7.2)

$= 0.04 \times 17{,}832 \times (2.19 + 14) \times 9.81$

$= 113$ kN-m.

Total overturning moment at the base of staging,

$M^* = \sqrt{M_i^{*2} + M_c^{*2}}$ (Section 4.7.3)

$= \sqrt{(924)^2 + (113)^2}$

$= 931$ kN-m.

1.10. Hydrodynamic Pressure

1.10.1. Impulsive Hydrodynamic Pressure

Impulsive hydrodynamic pressure on wall

$p_{iw}(y) = Q_{iw}(y)\, (A_h)_i\, \rho\, g\, h \cos\phi$

$Q_{iw}(y) = 0.866\, [1 - (y / h)^2]\, \tanh (0.866\, D / h)$

(Section 4.9.1(a))

Maximum pressure will occur at $\phi = 0$.

At base of wall, $y = 0$;

$Q_{iw}(y = 0) = 0.866[1-(0/3.0)^2] \times \tanh (0.866 \times 4.65 /3.0)$

$= 0.76$

Impulsive pressure at the base of wall,

$p_{iw}(y = 0) = 0.76 \times 0.06 \times 1{,}000 \times 9.81 \times 3.0 \times 1$

$= 1.41$ kN/m^2.

Impulsive hydrodynamic pressure on the base slab ($y = 0$)

$p_{ib} = 0.866 (A_h)_i \rho g h \sinh(0.866 x / L) / \cosh(0.866 l' / h)$

(Section 4.9.1(a))

$= 0.866 \times 0.06 \times 1{,}000 \times 9.81 \times 3.0 \times \sinh (0.866 \times 4.65 / (2 \times 3.0)) / \cosh (0.866 \times 4.65 / 2 \times 3.0)$

$= 0.95$ kN/m^2

1.10.2. Convective Hydrodynamic Pressure

Convective hydrodynamic pressure on wall,

$p_{cw} = Q_{cw}(y)\, (A_h)_c\, \rho\, g\, D\, [1 - 1/3 \cos^2\phi] \cos\phi$

$Q_{cw}(y) = 0.5625 \cosh (3.674\, y/D)/\cosh(3.674 h /D)$

(Section 4.9.2(a))

Maximum pressure will occur at $\phi = 0$.

At base of wall, $y = 0$;

$Q_{cw}(y = 0) = 0.5625 \times \cosh (0) / \cosh (3.674 \times 3.0 /4.65)$

$= 0.10$.

Convective pressure at the base of wall,

$p_{cw}(y = 0)$

$= 0.10 \times 0.04 \times 1{,}000 \times 9.81 \times 4.65 \times 0.67 \times 1$

$= 0.12$ kN/m^2

At $y = h$;

$Q_{cw}(y = h) = 0.5625$

Convective pressure at $y = h$,

$p_{cw}(y = h)$

$= 0.5625 \times 0.04 \times 1{,}000 \times 9.81 \times 4.65 \times 0.67 \times 1$

$= 0.69$ kN/m^2.

Convective hydrodynamic pressure on the base slab ($y = 0$)

$p_{cb} = Q_{cb}(x)\, (A_h)_c\, \rho\, g\, D$

$Q_{cb}(x) = 1.125[x/D - 4/3\ (x/D)^3]\ \text{sech}\ (3.674\ h/D)$

(Section 4.9.2(a))

$= 1.125[D/2D - 4/3\ (D/2D)^3]\ \text{sech}\ (3.674 \times 3/4.65)$

$= 0.07$

Convective pressure on top of base slab ($y = 0$)

$p_{cb} = 0.07 \times 0.04 \times 1{,}000 \times 9.81 \times 4.65$

$= 0.13\ \text{kN/m}^2$

1.11. Pressure Due to Wall Inertia

Pressure on wall due to its inertia,

$p_{ww} = (A_h)_i\, t\, \rho_m\, g$ (Section 4.9.5)

$= 0.06 \times 0.2 \times 25$

$= 0.32\ \text{kN/m}^2$.

This pressure is uniformly distributed along the wall height.

1.12. Pressure Due to Vertical Excitation

Hydrodynamic pressure on tank wall due to vertical ground acceleration,

$p_v = (A_v)\ [\rho\, g\, h\ (1 - y/h)]$

(Section 4.10.1)

$$A_v = \frac{2}{3}\left(\frac{Z}{2}\frac{I}{R}\frac{S_a}{g}\right)$$

$Z = 0.1$ (IS 1893(Part 1): Table 2; Zone II)

$I = 1.5$ (Table 1)

$R = 2.5$

Time period of vertical mode of vibration is recommended as 0.3 sec in Section 4.10.1. For 5 % damping, $S_a/g = 2.5$.

Hence,

$$A_v = \frac{2}{3} \times \left(\frac{0.1}{2} \times \frac{1.5}{2.5} \times 2.5\right)$$

$= 0.05$

At the base of wall, i.e., $y = 0$,

$p_v = 0.05 \times [\ 1 \times 9.81 \times 3 \times (\ 1 - 0/3\ )]$

$= 1.47\ \text{kN/m}^2$.

In this case, hydrodynamic pressure due to vertical ground acceleration is more than impulsive hydrodynamic pressure due to lateral excitation.

1.13. Maximum Hydrodynamic Pressure

Maximum hydrodynamic pressure,

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}^{\ 2} + p_v^{\ 2}}$$

(Section 4.10.2)

At the base of wall,

$$p = \sqrt{(1.41 + 0.32)^2 + 0.12^2 + 1.47^2}$$

$= 2.27\ \text{kN/m}^2$.

This maximum hydrodynamic pressure is about 8 % of hydrostatic pressure at base ($\rho\, g\, h = 1{,}000 \times 9.81 \times 3.0 = 29.43\ \text{kN/m}^2$).

In practice, container of tank is designed by working stress method. When earthquake forces are considered, permissible stresses are increased by 33%. Hence, hydrodynamic pressure in this case does not affect container design.

1.14. Sloshing Wave Height

Maximum sloshing wave height,

$d_{max} = (A_h)_c\ R\ \ D/2$ (Section 4.11)

$= 0.04 \times 2.5 \times 4.65 / 2$

$= 0.23$ m.

Height of sloshing wave is less than free board of 0.3 m.

1.15. Analysis for Tank Empty Condition

For empty condition, tank will be considered as single degree of freedom system as described in Section 4.7.4.

Mass of empty container + one third mass of staging, $m_s = 63{,}799$ kg.

Stiffness of staging, $K_s = 6{,}060$ kN/m.

1.15.1. Time Period

Time period of impulsive mode,

$$T = T_i = 2\pi\sqrt{\frac{m_s}{K_s}}$$

$$= 2\pi\sqrt{\frac{63{,}799}{60{,}60{,}000}}$$

$= 0.65$ sec.

Empty tank will not have convective mode of vibration.

1.15.2. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient corresponding to impulsive time period T_i,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.1$ (IS 1893(Part 1): Table 2; Zone II)

$I = 1.5$ (Table 1)

$R = 2.5$ (Table 2)

Here, $T_i = 0.65$ sec,

Site has soft soil,

Damping = 5%,

Hence, $(S_a/g)_i = 2.5$ (IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.1}{2} \times \frac{1.5}{2.5} \times 2.5 = 0.08.$$

1.15.3. Base Shear

Total base shear,

$V = V_i = (A_h)_i\, m_s\, g$ (Section 4.6.2)

$= 0.08 \times 63{,}799 \times 9.81$

$= 50$ kN.

1.15.4. Base Moment

Total base moment,

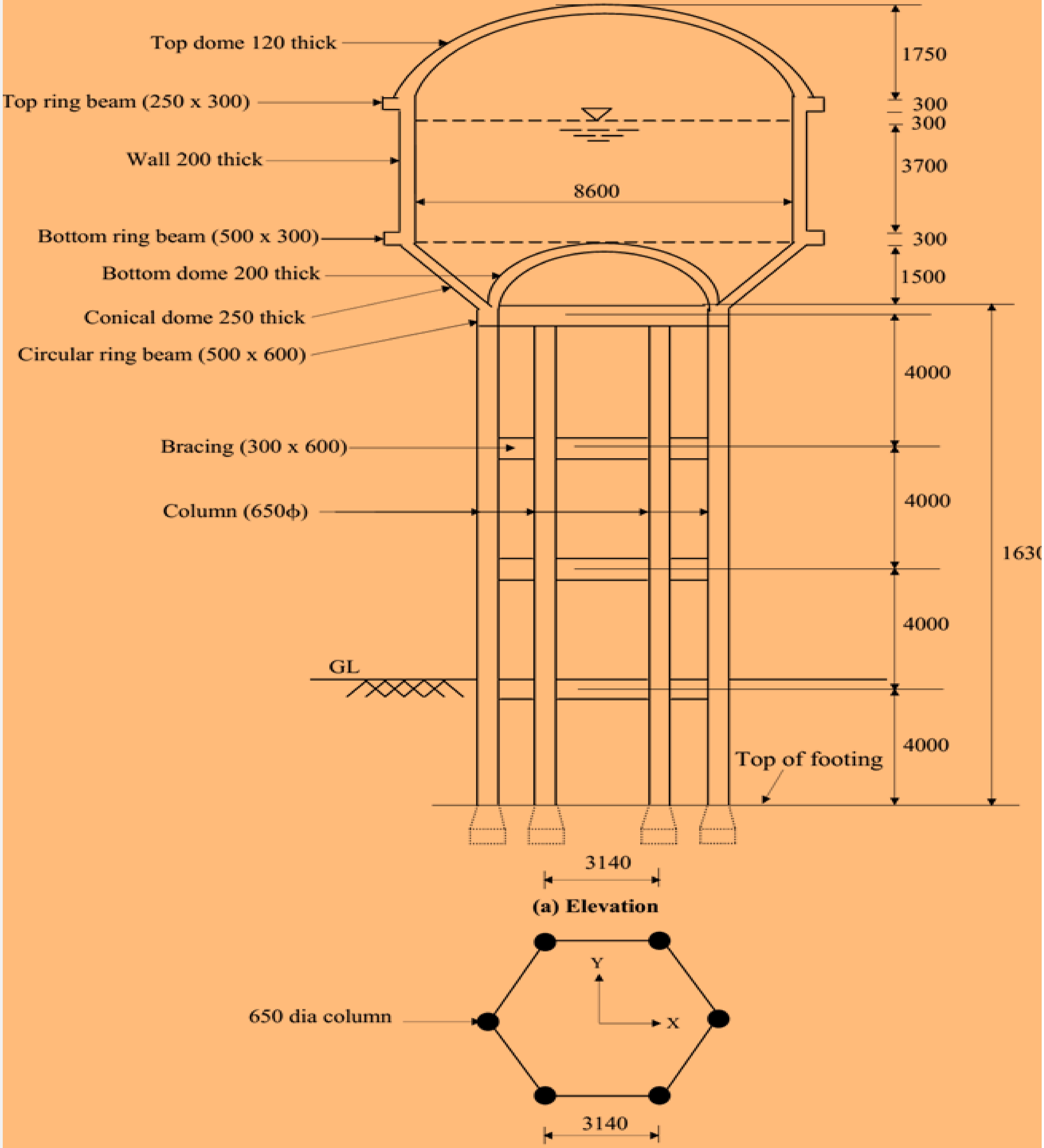
$M^* = (A_h)_i\; m_s\, h_{cg}\; g$ (Section 4.7.3)

$= 0.08 \times 63{,}799 \times 15.18 \times 9.81$

$= 760$ kN-m

Since total base shear (60 kN) and base moment (931 kN-m) in tank full condition are more than that total base shear (50 kN) and base moment (760 kN-m) in tank empty condition, design will be governed by tank full condition.

Example 2 – Elevated Intze Tank Supported on 6 Column RC Staging



(a) Elevation

(b) Plan of staging

(All dimensions in mm)

Figure 2.1: Details of tank geometry

Example 2 – Elevated Intze Tank Supported on 6 Column RC Staging

2. Problem Statement:

An intze shape water container of 250 m^3 capacity is supported on RC staging of 6 columns with horizontal bracings of 300 x 600 mm at three levels. Details of staging configuration are shown in Figure 2.1. Staging conforms to ductile detailing as per IS 13920. Grade of concrete and steel are M20 and Fe415, respectively. Tank is located on hard soil in seismic zone IV. Density of concrete is 25 kN/m^3. Analyze the tank for seismic loads.

Solution:

Tank must be analysed for tank full and empty conditions.

2.1. Preliminary Data

Details of sizes of various components and geometry are shown in Table 2.1 and Figure 2.1.

Table 2.1 Sizes of various components

Component	Size (mm)
Top Dome	120 thick
Top Ring Beam	250 x 300
Cylindrical Wall	200 thick
Bottom Ring Beam	500 x 300
Circular Ring Beam	500 x 600
Bottom Dome	200 thick
Conical Dome	250 thick
Braces	300 x 600
Columns	650 dia.

2.2. Weight calculations

Table 2.2 Weight of various components

Components	Calculations	Weight (kN)
Top Dome	Radius of dome, $r_1 = [((8.8/2)^2 / 1.69) + 1.69)] / 2 = 6.57$ $2 \times \pi \times 6.57 \times 1.69 \times (0.12 \times 25)$	209.3
Top Ring Beam	$\pi \times (8.6 + 0.25) \times 0.25 \times 0.30 \times 25$	52.1
Cylindrical Wall	$\pi \times 8.8 \times 0.20 \times 4.0 \times 25$	552.9
Bottom Ring Beam	$\pi \times (8.6 + 0.5) \times 0.5 \times 0.30 \times 25$	107.2
Circular Ring Beam	$\pi \times 6.28 \times 0.50 \times 0.60 \times 25$	148
Bottom Dome	Radius of dome, $r_2 = [(6.28/2)^2 / 1.40) + 1.40] / 2 = 4.22$ $2 \times \pi \times 4.22 \times 1.40 \times 0.20 \times 25$	185.6
Conical Dome	Length of Cone, $L_c = (1.65^2 + 1.41^2)^{1/2} = 2.17$ $\pi \times ((8.80 + 6.28) / 2.0) \times 2.17 \times 0.25 \times 25$	321.3
Water	$[(\pi \times 8.6^2 \times 3.7 /4) + (\pi \times 1.5(8.6^2 + 5.63^2 + (8.6 \times 5.63)) / 12$ $- (\pi \times 1.3^2 \times (3 \times 4.22 - 1.5) / 3)] \times 9.81$	2,508
Columns	$\pi \times (0.65)^2 \times 15.7 \times 6 \times 25 / 4$	782
Braces	$3.14 \times 0.30 \times 0.60 \times 3 \times 6 \times 25$	254

Note: - i) Wherever floor finish and plaster is provided, their weights should be included in the weight calculations.

ii) No live load is considered on roof slab and gallery for seismic load computations.

iii) Water load is considered as dead load.

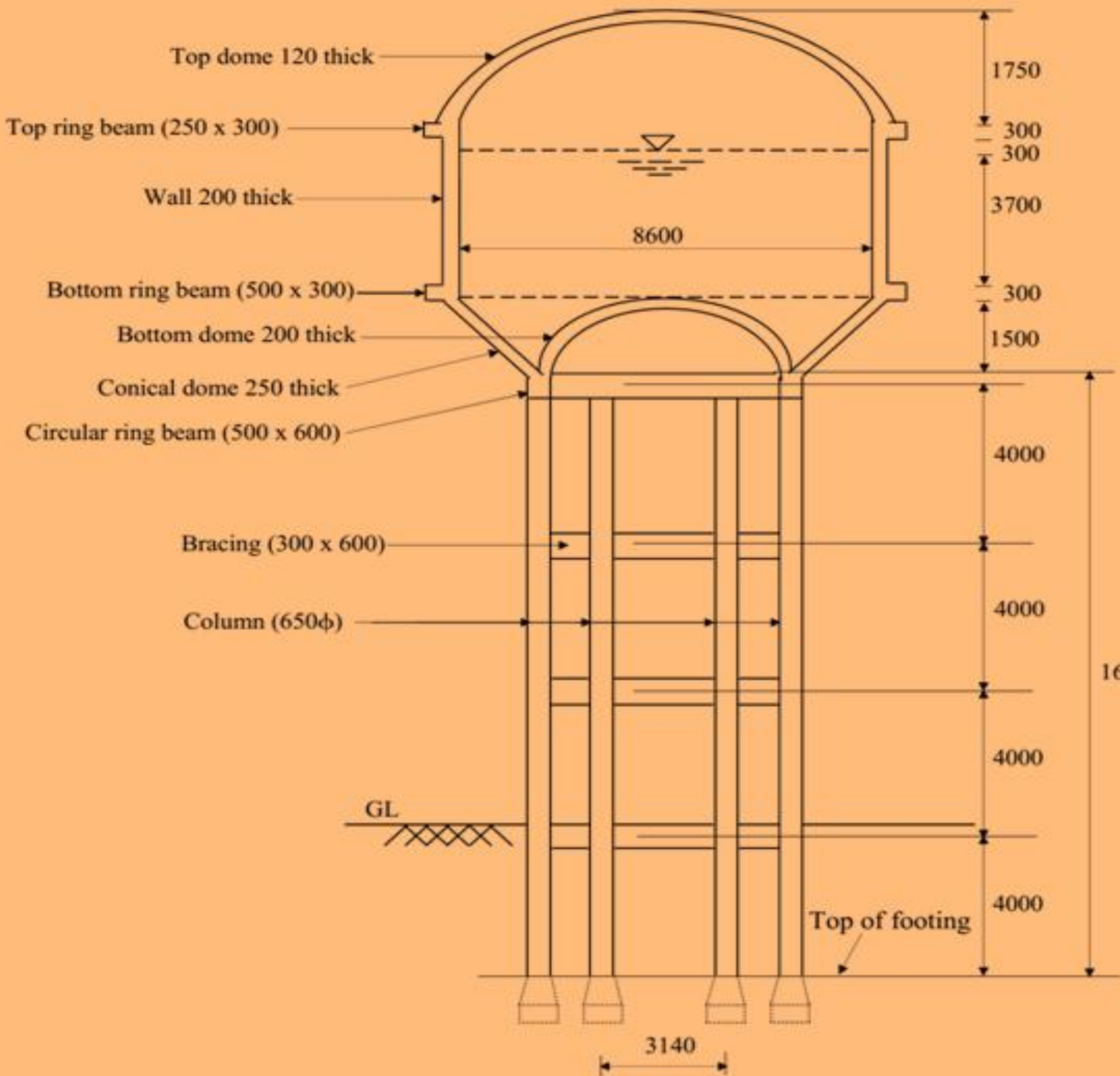
iv) For seismic analysis, free board is not included in depth of water.

From Table 2.2,

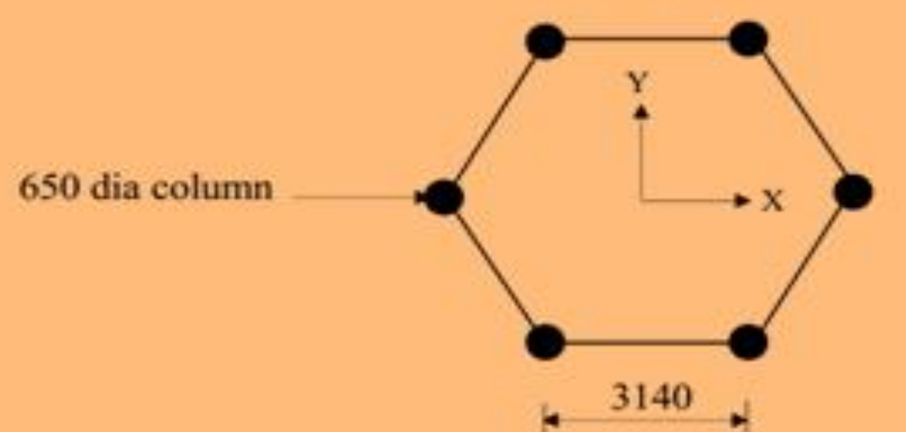
Weight of empty container = 209.3 + 52.1+ 552.9 + 107.2 + 148 + 185.6 + 321.3 = 1,576 kN

Weight of staging = 782 + 254 = 1,036 kN

Hence, weight of empty container + one third weight of staging = 1,576 + 1,036 / 3 = 1,921 kN



(a) Elevation



(b) Plan of staging

(All dimensions in mm)

Figure 2.1: Details of tank geometry

2.3. Center of Gravity of Empty Container

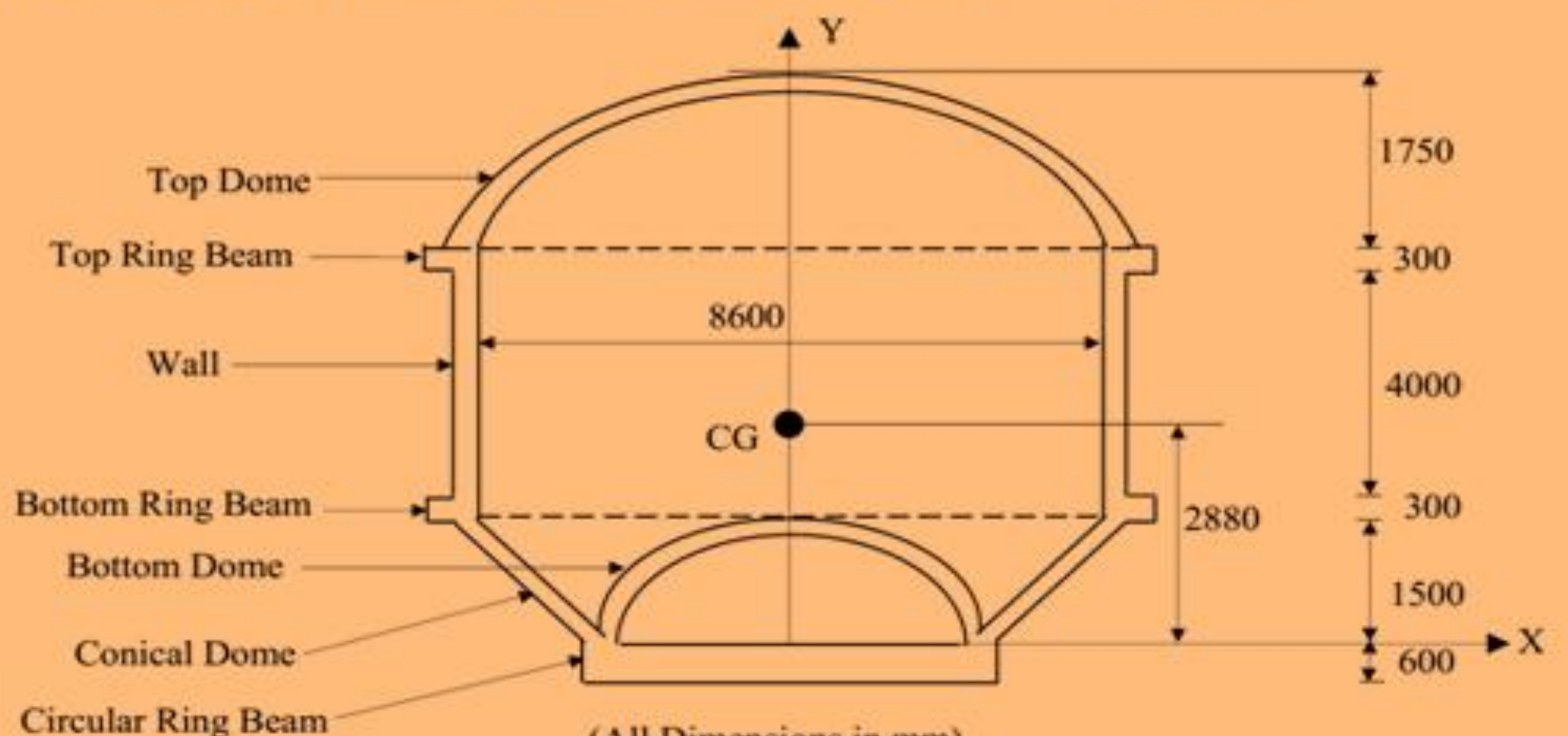
Components of empty container are: top dome, top ring beam, cylindrical wall, bottom ring beam, bottom dome, conical dome and circular ring beam. With reference to Figure 2.2,

Height of CG of empty container above top of circular ring beam,

$$= [(209.3 \times 7.22) + (52.1 \times 5.9) + (552.9 \times 3.8) + (107.2 \times 1.65) + (321.3 \times 1) + (185.6 \times 0.92) - (148 \times 0.3)] / 1,576$$

$$= 2.88 \text{ m}$$

Height of CG of empty container from top of footing, $h_{cg} = 16.3 + 2.88 = 19.18$ m.



(All Dimensions in mm)

Figure 2.2 Details of tank container

2.4. Parameters of Spring Mass Model

Total weight of water = 2,508 kN = 25,08,000 N.

Volume of water = 2,508 / 9.81 = 255.65 m^3

Mass of water, m = 2,55,658 kg.

Inner diameter of tank, D = 8.6 m.

For obtaining parameters of spring mass model, an equivalent circular container of same volume and diameter equal to diameter of tank at top level of liquid will be considered.

(Section 4.2.3)

Let h be the height of equivalent circular cylinder,

$\pi (D/2)^2 h = 255.65$

$h = 255.65 / [\pi \times (8.6/2)^2] = 4.4$ m

For $h/D = 4.4 / 8.6 = 0.51$,

$m_i / m = 0.55$;

$m_i = 0.55 \times 2,55,658 = 1,40,612$ kg

$m_c/m = 0.43$;

$m_c = 0.43 \times 2,55,658 = 1,09,933$ kg

$h_i / h = 0.375$; $h_i = 0.375 \times 4.4 = 1.65$ m

$h_i^* / h = 0.78$; $h_i^* = 0.78 \times 4.4 = 3.43$ m

$h_c / h = 0.61$; $h_c = 0.61 \times 4.4 = 2.68$ m

$h_c^* / h = 0.78$; $h_c^* = 0.78 \times 4.4 = 3.43$ m.

(Section 4.2.1)

About 55% of liquid mass is excited in impulsive mode while 43% liquid mass participates in convective mode. Sum of impulsive and convective mass is 2,50,545 kg which is about 2 % less than the total mass of liquid.

Mass of empty container + one third mass of staging,

$m_s = (1,576 + 1,036 / 3) \times (1,000 / 9.81)$

$= 1,95,821$ kg.

2.5. Lateral Stiffness of Staging

Lateral stiffness of staging is defined as the force required to be applied at the CG of tank so as to get a corresponding unit deflection. As per Section 4.3.1.3, CG of tank is the combined CG of empty container and impulsive mass. However, in this example, CG of tank is taken as CG of empty container.

Finite element software is used to model the staging (Refer Figure 2.3). Modulus of elasticity for M20 concrete is obtained as 5,000 $\sqrt{f_{ck}}$ = 5,000 x $\sqrt{20}$ = 22,360 MPa or 22.36 x 10^6 kN/m^2. Since container portion is quite rigid, a rigid link is assumed from top of staging to the CG of tank. In FE model of staging, length of rigid link is = 2.88 + 0.3 = 3.18 m.

From the analysis deflection of CG of tank due to an arbitrary 10 kN force is obtained as 5.616E-04 m.

Thus, lateral stiffness of staging, K_s

= 10 / (5.616E-04) = 17,800 kN/m

Stiffness of this type of staging can also be obtained using method described by Sameer and Jain (1992).

Here analysis of staging is being performed for earthquake loading in X-direction. However, for some members of staging, earthquake loading in Y-direction will be critical, as described in Section 4.8.2.

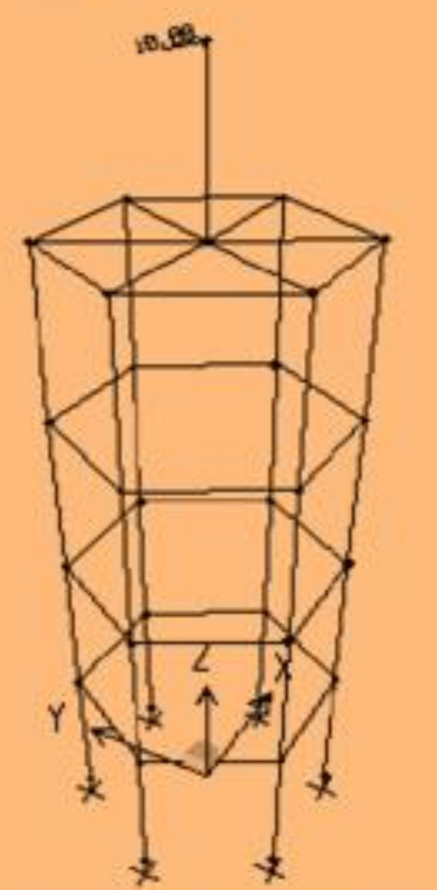


Figure 2.3 FE model of staging

2.6. Time Period

Time period of impulsive mode,

$$T_i = 2\pi\sqrt{\frac{m_i + m_s}{K_s}} \qquad \text{(Section 4.3.1.3)}$$

$$= 2\pi\sqrt{\frac{1,40,612 + 1,95,821}{178.0\times10^5}}$$

= 0.86 sec.

Time period of convective mode,

$$T_c = C_c\sqrt{\frac{D}{g}}$$

For $h / D = 0.51$, $C_c = 3.35$

(Section 4.3.2.2 (a))

Thus, $T_c = 3.35\sqrt{\frac{8.6}{9.81}} = 3.14$ sec.

2.7. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

Since staging has special moment resisting frames (SMRF), R is taken as 2.5

(Table 2)

Here, $T_i = 0.86$ sec,

Site has hard soil,

Damping = 5%, (Section 4.4)

Hence, $(S_a/g)_i = 1.16$

(IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.24}{2}\times\frac{1.5}{2.5}\times1.16 = 0.084$$

Design horizontal seismic coefficient for convective mode,

$$(A_h)_c = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_c \qquad \text{(Sections 4.5 and 4.5.1)}$$

Where,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

For convective mode, value of R is taken same as that for impulsive mode as per Section 4.5.1.

Here, $T_c = 3.14$ sec,

Site has hard soil,

Damping = 0.5%, (Section 4.4)

Hence, as per Section 4.5.3 and IS 1893(Part 1): 2002, Figure 2

$(S_a/g)_c = 1.75 \times 0.318 = 0.56$

Multiplying factor of 1.75 is used to obtain S_a/g values for 0.5% damping from that for 5% damping.

(Section 4.5.4)

$$(A_h)_c = \frac{0.24}{2} \times \frac{1.5}{2.5} \times 0.56 = 0.040$$

2.8. Base Shear

Base shear at the bottom of staging, in impulsive mode,

$V_i = (A_h)_i \ (m_i + m_s)\, g$ (Section 4.6.2)

$= 0.084 \times (1{,}40{,}612 + 1{,}95{,}821) \times 9.81$

$= 277$ kN

Similarly, base shear in convective mode,

$V_c = (A_h)_c\, m_c\, g$ (Section 4.6.2)

$= 0.040 \times 1{,}09{,}933 \times 9.81$

$= 43$ kN

Total base shear at the bottom of staging,

$V = \sqrt{V_i^2 + V_c^2}$ (Section 4.6.3)

$= \sqrt{(277)^2 + (43)^2}$

$= 280$ kN.

It may be noted that total lateral base shear is about 6 % of total seismic weight (4,429 kN) of tank.

2.9. Base Moment

Overturning moment at the base of staging in impulsive mode,

$M_i^* = (A_h)_i [\, m_i\, (\, h_i^* + h_s\,) + m_s\, h_{cg}\,]\, g$

(Section 4.7.2)

$= 0.084 \times [1{,}40{,}612 \times (3.43 + 16.3)$

$+ (1{,}95{,}821 \times 19.18)] \times 9.81$

$= 5{,}381$ kN-m

Similarly, overturning moment in convective mode,

$M_c^* = (A_h)_c\, m_c\, (\, h_c^* + h_s\,)\, g$

(Section 4.7.2)

$= 0.040 \times 1{,}09{,}933 \times (3.43 + 16.3) \times 9.81$

$= 852$ kN-m

Total overturning moment,

$M^* = \sqrt{M_i^{*2} + M_c^{*2}}$ (Section 4.7.3)

$= \sqrt{(5{,}381)^2 + (852)^2}$

$= 5{,}448$ kN-m.

Note: Hydrodynamic pressure calculations will be similar to those shown in Example 1 and hence are not included here.

2.10. Sloshing Wave Height

$d_{max} = (\, A_h)_c\, R\ \ D/2$ (Section 4.11)

$= 0.040 \times 2.5 \times 8.6 / 2$

$= 0.43$ m.

2.11. Analysis for Tank Empty Condition

For empty condition, tank will be considered as single degree of freedom system as described in Section 4.7.4.

Mass of empty container + one third mass of staging, $m_s = 1{,}95{,}821$ kg.

Stiffness of staging, $K_s = 17{,}800$ kN/m.

2.11.1. Time Period

Time period of impulsive mode,

$$T = T_i = 2\pi\sqrt{\frac{m_s}{K_s}}$$

$$= 2\pi\sqrt{\frac{1{,}95{,}821}{178.0 \times 10^5}} = 0.66 \text{ sec}$$

Empty tank will not convective mode of vibration.

2.11.2. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient corresponding to impulsive time period T_i,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$ (Section 4.5)

Where,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

$R = 2.5$ (Table 2)

Here, $T_i = 0.66$ sec,

Site has hard soil,

Damping = 5%,

Hence, $(S_a/g)_i = 1.52$

(IS 1893(Part 1): 2002Figure 2)

$$(A_h)_i = \frac{0.24}{2} \times \frac{1.5}{2.5} \times 1.52 = 0.11.$$

2.11.3. Base Shear

Total base shear,

$V = V_i = (A_h)_i\, m_s\, g$ (Section 4.6.2)

$= 0.12 \times 1{,}95{,}821 \times 9.81$

$= 211$ kN.

2.11.4. Base Moment

Total base moment,

$M^* = (A_h)_i\ m_s\, h_{cg}\ g$ (Section 4.7.3)

$= 0.11 \times 1{,}95{,}821 \times 19.18 \times 9.81$

$= 4{,}053$ kN-m

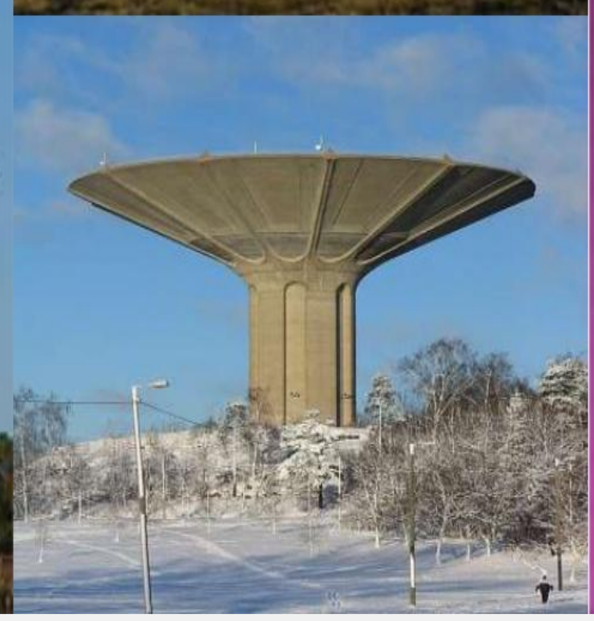
Since total base shear (280 kN) and base moment (5,448 kN-m) in tank full condition are more than base shear (211 kN) and base moment (4,053 kN-m) in tank empty condition, design will be governed by tank full condition.

Example 3 - Elevated Intze Tank Supported on RC Shaft






BWSC



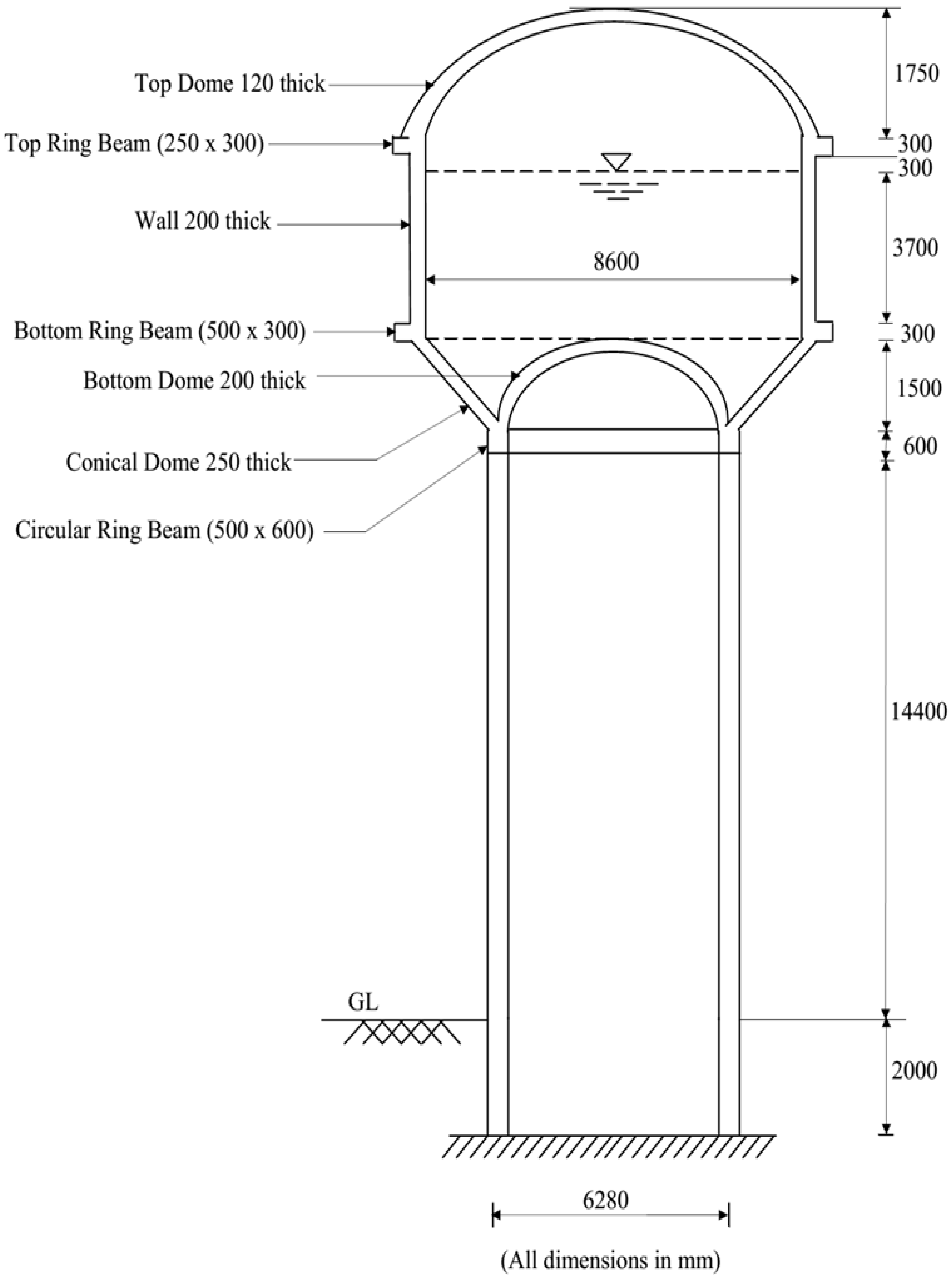


Figure 3.1 Details of tank geometry

Example 3 –Elevated Intze Tank Supported on RC Shaft

3. Problem Statement:

Intze container of previous example is considered to be supported on 15 m high hollow RC shaft with reinforcement in two curtains. Grade of concrete and steel are M20 and Fe415, respectively. Site of the tank has hard soil in seismic zone IV. Density of concrete is 25 kN/m^3. Analyze the tank for seismic loads.

Solution:

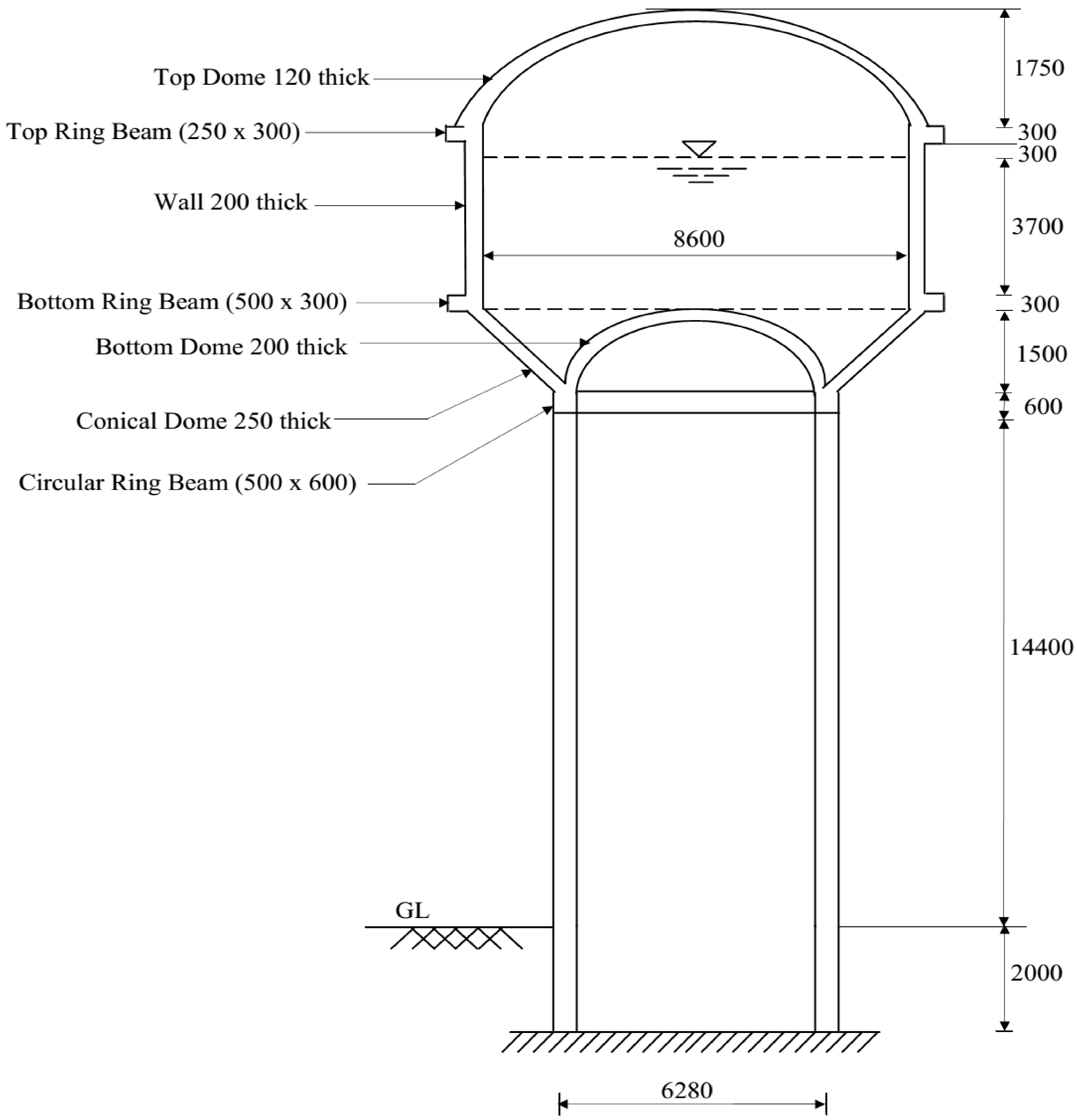
Tank will be analysed for tank full and empty conditions.

3.1. Preliminary Data

Container data is same as one given in previous example. Additional relevant data is listed below:

1. Thickness of shaft = 150 mm.
2. Weight of shaft = π x 6.28 x 0.15 x 16.4 x 25 = 1,213 kN
3. Weight of empty container + one third weight of staging = 1,576 + 1,213 / 3 = 1,980 kN
4. Since staging height is 17 m from footing level, height of CG of empty container from top of footing,

   h_{cg} = 17 + 2.88 = 19.88 m



(All dimensions in mm)

Figure 3.1 Details of tank geometry

3.2. Parameters of Spring Mass Model

Total weight of water = 2,508 kN = 25,08,000 N.

Volume of water = 2,508 / 9.81 = 255.66 m^3

Mass of water, m = 2,55,658 kg.

Inner diameter of tank, D = 8.6 m.

For obtaining parameters of spring mass model, an equivalent circular container of same volume and diameter equal to diameter of tank at top level of liquid will be considered.

(Section 4.2.3)

Let h be the height of equivalent circular cylinder,

$\pi (D/2)^2 h = 255.66$

$h = 255.66 / [\pi \times (8.6 / 2)^2] = 4.4$ m

For $h / D = 4.4 / 8.6 = 0.51$,

$m_i / m = 0.55$;

$m_i = 0.55 \times 2{,}55{,}658 = 1{,}40{,}612$ kg

$m_c / m = 0.43$;

$m_c = 0.43 \times 2{,}55{,}658 = 1{,}09{,}933$ kg

$h_i / h = 0.375$; $h_i = 0.375 \times 4.4 = 1.65$ m

$h_i^* / h = 0.78$; $h_i^* = 0.78 \times 4.4 = 3.43$ m

$h_c / h = 0.61$; $h_c = 0.61 \times 4.4 = 2.68$ m

$h_c^* / h = 0.78$; $h_c^* = 0.78 \times 4.4 = 3.43$ m.

(Section 4.2.1)

Note that about 55% of liquid is excited in impulsive mode while 43% participates in convective mode. Sum of impulsive and convective mass is about 2% less than total mass of liquid.

Mass of empty container + one third mass of staging,

$m_s = (1{,}576 + 1{,}213 / 3) \times (1{,}000 / 9.81)$

$= 2{,}01{,}869$ kg.

3.3. Lateral Stiffness of Staging

Here, shaft is considered as cantilever of length 16.4 m. This is the height of shaft from top of footing upto bottom of circular ring beam.

Lateral Stiffness, $K_s = 3\, E\, I / L^3$

Where,

E = Modulus of elasticity = $5{,}000\ \sqrt{f_{ck}}$

$= 5{,}000 \times \sqrt{20} = 22{,}360$ N/mm^2

$= 22.36 \times 10^6$ kN/ m^2

I = Moment of inertia of shaft cross section

$= \pi \times (6.43^4 - 6.13^4) / 64$

$= 14.59\ m^4$

L = Height of shaft

= 16.4 m

Thus,

Lateral Stiffness = $3 \times 22{,}360 \times 10^6 \times 14.59 / 16.4^3$

$= 2.22 \times 10^8$ N/m

NOTE:- Here, only flexural deformations are considered in the calculation of lateral stiffness and the effect of shear deformation is not included. If the effect of shear deformations is included then the lateral stiffness is given by:

$$K_S = \frac{1}{\dfrac{L^3}{3EI} + \dfrac{L}{\kappa' AG}}$$

Where, G is shear modulus, A is cross sectional area and κ' is shape factor.

3.4. Time Period

Time period of impulsive mode,

$$T_i = 2\pi \sqrt{\frac{m_i + m_s}{K_s}}$$ (Section 4.3.1.3)

$$= 2\pi \sqrt{\frac{1{,}40{,}612 + 2{,}01{,}869}{2.22 \times 10^8}}$$

= 0.25 sec.

Time period of convective mode,

$$T_c = C_c \sqrt{\frac{D}{g}}$$

For $h / D = 0.51$, $C_c = 3.35$

(Section 4.3.2.2 (a))

Thus, $T_c = 3.35 \sqrt{\dfrac{8.6}{9.81}} = 3.14$ sec.

3.5. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

Shaft is considered to have reinforcement in two curtains both horizontally and vertically. Hence R is taken as 1.8. (Table 2)

Here, $T_i = 0.25$ sec,

Site has hard soil,

Damping = 5%, (Section 4.4)

Hence, $(S_a/g)_i = 2.5$

(IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.24}{2} \times \frac{1.5}{1.8} \times 2.5 = 0.25$$

Design horizontal seismic coefficient for convective mode,

$$(A_h)_c = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_c$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

For convective mode, value of R is taken same as that for impulsive mode as per Section 4.5.1.

Here, $T_c = 3.14$ sec,

Site has hard soil,

Damping = 0.5%, (Section 4.4)

Hence, as per Section 4.5.3 and IS 1893(Part 1): 2002, Figure 2

$(S_a/g)_c = 1.75 \text{ x } 0.318 = 0.56$

Multiplying factor of 1.75 is used to obtain S_a/g values for 0.5% damping from that for 5% damping.

(Section 4.5.4)

$$(A_h)_c = \frac{0.24}{2} \times \frac{1.5}{1.8} \times 0.56 = 0.06$$

3.6. Base Shear

Base shear at the bottom of staging, in impulsive mode,

$V_i = (A_h)_i \ (m_i + m_s)\, g$

(Section 4.6.2)

= 0.25 x (1,40,612 + 2,01,869) x 9.81

= 840 kN

Similarly, base shear in convective mode,

$V_c = (A_h)_c\, m_c\, g$ (Section 4.6.2)

= 0.06 x 1,09,933 x 9.81

= 65 kN

Total base shear at the bottom of staging,

$V = \sqrt{V_i^2 + V_c^2}$ (Section 4.6.3)

$= \sqrt{(840)^2 + (65)^2}$

= 843 kN.

It may be noted that total lateral base shear is about 19% of total seismic weight (4,488 kN) of tank.

3.7. Base Moment

Overturning moment at the base of staging in impulsive mode,

$M_i^* = (A_h)_i\,[\, m_i\,(\, h_i^* + h_s\,) + m_s\, h_{cg}\,]\, g$

(Section 4.7.2)

= 0.25 x [1, 40,612 x (3.43 + 17)

+ (2,01,869 x 19.88)] x 9.81

= 16,888 kN-m

Similarly, overturning moment in convective mode,

$M_c^* = (A_h)_c\, m_c\, (h_c^* + h_s)\, g$

(Section 4.7.2)

= 0.06 x 1,09,933 x (3.43 + 17) x 9.81

= 1,322 kN-m

Total overturning moment,

$M^* = \sqrt{M_i^{*2} + M_c^{*2}}$ (Section 4.7.3)

$= \sqrt{(16{,}888)^2 + (1{,}322)^2}$

= 16,940 kN-m.

3.8. Sloshing Wave Height

Maximum sloshing wave height,

$d_{max} = (A_h)_c\, R\, D / 2$ (Section 4.11)

$= 0.06 \times 1.8 \times 8.6 / 2$

$= 0.46$ m

Note – Hydrodynamic pressure calculations will be similar to those shown in Example 1, hence are not repeated.

3.9. Analysis for Tank Empty Condition

For empty condition, tank will be considered as single degree of freedom system as described in Section 4.7.4.

Mass of empty container + one third mass of staging, m_s = 2,01,869 kg

Stiffness of staging, $K_s = 2.22 \times 10^8$ N/m

3.9.1. Time Period

Time period of impulsive mode,

$$T_i = 2\pi\sqrt{\frac{m_s}{K_s}}$$

$$= 2\pi\sqrt{\frac{2{,}01{,}869}{2.22\times10^8}}$$

= 0.19 sec.

Empty tank will not have convective mode of vibration.

3.9.2. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient corresponding to impulsive time period T_i,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$ (Section 4.5)

Where,

$Z = 0.24$

(IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

$R = 1.8$ (Table 2)

Here, $T_i = 0.19$ sec,

Site has hard soil,

Damping = 5%

Hence, $(S_a/g)_i = 2.5$

(IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.24}{2}\times\frac{1.5}{1.8}\times 2.5 = 0.26$$

3.9.3. Base Shear

Total base shear,

$V = V_i = (A_h)_i\, m_s\, g$ (Section 4.6.2)

$= 0.25 \times 2{,}01{,}869 \times 9.81$

$= 495$ kN

3.9.4. Base Moment

Total base moment,

$M^* = (A_h)_i\; m_s\, h_{cg}\; g$ (Section 4.7.3)

$= 0.25 \times 2{,}01{,}869 \times 19.88 \times 9.81$

$= 9{,}842$ kN-m

For this tank, since total base shear in tank full condition (843 kN) is more than that in tank empty condition, (495 kN) design will be governed by tank full condition.

Similarly, for base moment, tank full condition is more critical than in tank empty condition.

Note: Pressure calculations are not shown for this tank.

Example 4: Ground Supported Circular Steel Tank

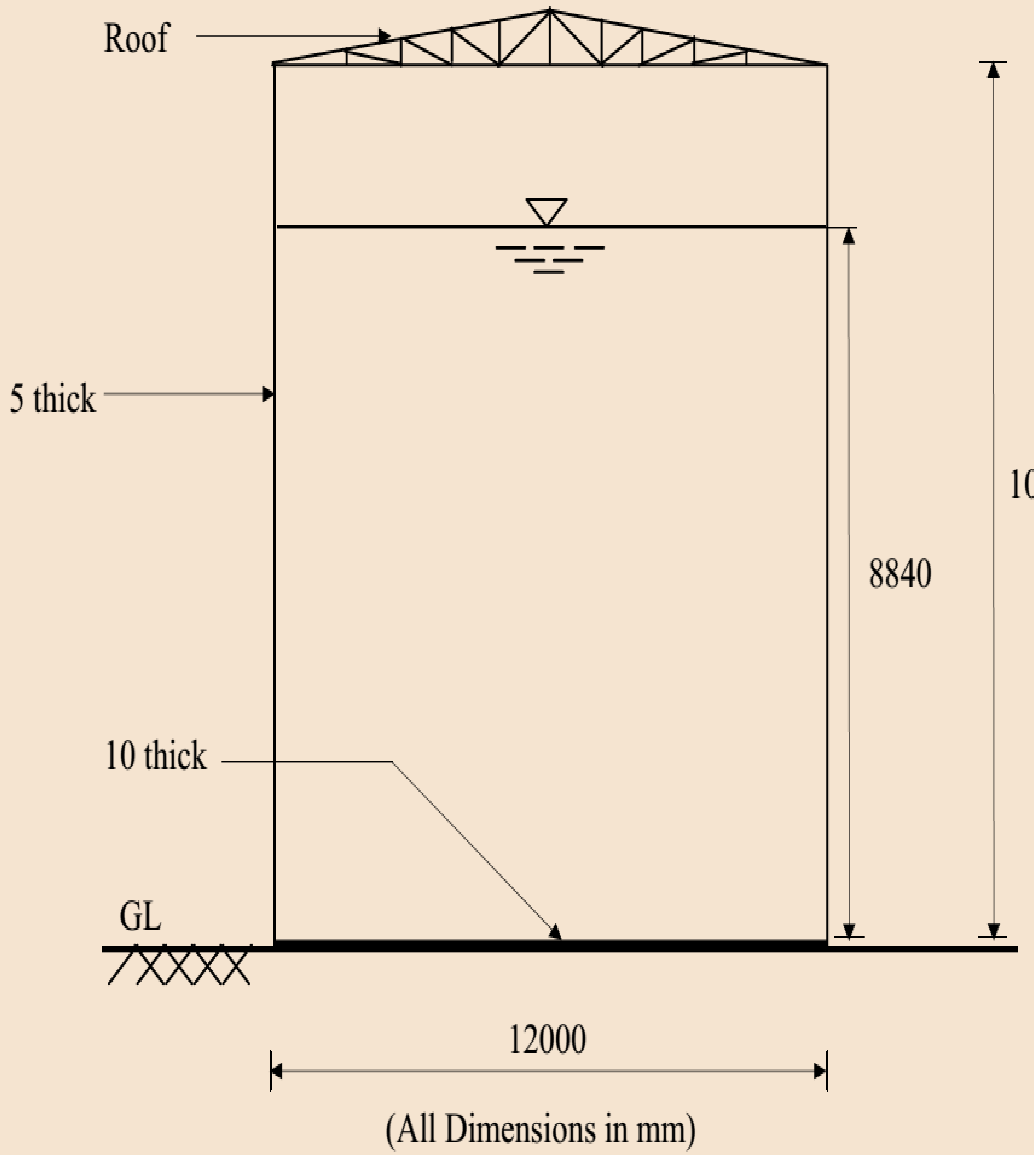


Figure 4.1 Sectional elevation of tank

Example 4: Ground Supported Circular Steel Tank

4. Problem Statement:

A ground supported cylindrical steel tank with 1,000 m^3 capacity has inside diameter of 12 m, height of 10.5 m and wall thickness is 5 mm. Roof of tank consists of stiffened steel plates supported on roof truss. Tank is filled with liquid of specific gravity 1.0. Tank has a base plate of 10 mm thickness supported on hard soil in zone V. Density of steel plates is 78.53 kN/m^3. Analyze the tank for seismic loads.

Solution:

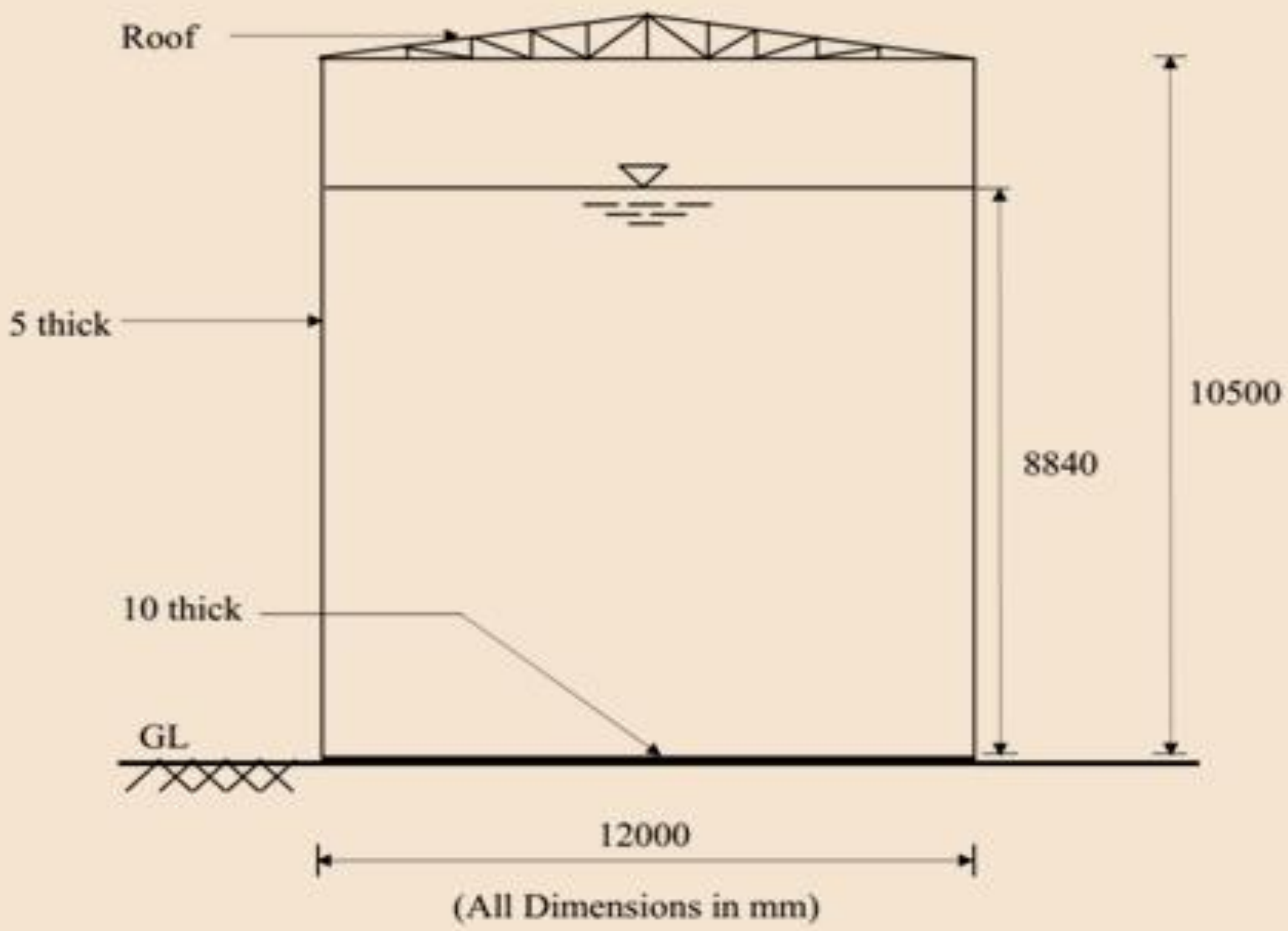


Figure 4.1 Sectional elevation of tank

4.1. Weight Calculations

Weight of tank wall

$= \pi \times (12 + 0.005) \times 0.005 \times 78.53 \times 10.5$

$= 156$ kN

Mass of tank wall, m_w

$= 156 \times 1{,}000 / 9.81$

$= 15{,}902$ kg

Weight of base plate

$= \pi \times (6.005)^2 \times 0.01 \times 78.53$

$= 89$ kN

Mass of base plate, m_b

$= 89 \times 1{,}000 / 9.81$

$= 9{,}072$ kg.

Volume of liquid = 1,000 m^3.

Weight of liquid = 9,810 kN

Mass of liquid, m = 10,00,000 kg

Assuming that roof of tank is a plate of 5 mm.

Weight of roof = 50 kN

Mass of roof, m_t

$= 50 \times 1{,}000 / 9.81$

$= 5{,}097$ kg

4.2. Parameters of Spring Mass Model

$h = 8.84$ m; $D = 12$ m

For $h / D = 8.84 / 12 = 0.74$,

$m_i / m = 0.703$;

$m_i = 0.703 \times 10{,}00{,}000 = 7{,}03{,}000$ kg

$m_c / m = 0.309$

m_c = 0.309 x 10,00,000 = 3,09,000 kg

h_i / h = 0.375 ; h_i = 0.375 x 8.84 = 3.32 m

h_c / h = 0.677 ; h_c = 0.677 x 8.84 = 5.98 m

h_i^* / h = 0.587 ; h_i^* = 0.587 x 8.84 = 5.19 m

h_c^* / h = 0.727 ; h_c^* = 0.727 x 8.84 = 6.43 m

(Section 4.2.1.2)

Note that about 70% of liquid is excited in impulsive mode while 30% participates in convective mode. Total liquid mass is about 1% less than sum of impulsive and convective masses.

4.3. Time Period

Time period of impulsive mode,

$$T_i = \frac{C_i h \sqrt{\rho}}{\sqrt{(t/D)}\sqrt{E}}$$

Where,

h = Depth of liquid = 8.84 m;

ρ = Mass density of liquid = 1,000 kg/m^3;

t = Thickness of wall = 0.005 m;

D = Inside diameter of tank = 12 m;

E = Young's modulus for steel = 2 x 10^{11} N/m^2

For h / D = 0.74, C_i = 4.23

(Section 4.3.1.1)

$$= \frac{4.23 \times 8.84 \times \sqrt{1{,}000}}{\sqrt{(0.005/12)} \times \sqrt{2\times10^{11}}}$$

= 0.13 sec.

Time period of convective mode,

$$T_c = C_c \sqrt{\frac{D}{g}}$$

For h / D = 0.74, C_c = 3.29

(Section 4.3.2.2(a))

$$T_c = 3.29\sqrt{\frac{12}{9.81}} = 3.64 \text{ sec.}$$

4.4. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Sections 4.5 and 4.5.1)

Where,

Z = 0.36 (IS 1893(Part 1): Table 2; Zone V)

I = 1.5 (Table 1)

R = 2.5 (Table 2)

This steel tank has anchored base, hence R is taken as 2.5.

Here, T_i = 0.13 sec,

Site has hard soil,

Damping = 5%, (Section 4.4)

Hence, S_a /g = 2.5 x 1.4 = 3.5

(IS 1893(Part 1): Figure 2)

Multiplying factor of 1.4 is used to obtain S_a /g for 2% damping from that for 5% damping.

(IS 1893(Part 1): Table 3)

$$(A_h)_i = \frac{0.36}{2} \times \frac{1.5}{2.5} \times 3.5 = 0.38$$

Design horizontal seismic coefficient for convective mode,

$$(A_h)_c = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_c$$

(Sections 4.5 and 4.5.1)

Where,

Z = 0.36 (IS 1893(Part 1): Table 2; Zone V)

I = 1.5 (Table 1)

R = 2.5

For convective mode, value of R is taken same as that for impulsive mode, as per Section 4.5.1.

Here, T_c = 3.64 sec,

Site has hard soil,

Damping = 0.5%, (Section 4.4)

Hence, as per Section 4.5.3 and IS 1893(Part 1): 2002, Figure 2

$(S_a/g)_c$ = 1.75 x 0.275 = 0.48

Multiplying factor of 1.75 is used to obtain S_a /g values for 0.5 % damping from that for 5 % damping.

(Section 4.5.4)

$$(A_h)_c = \frac{0.36}{2} \times \frac{1.5}{2.5} \times 0.48 = 0.05$$

4.5. Base Shear

Base shear at the bottom of wall in impulsive mode,

$V_i = (A_h)_i (m_i + m_w + m_t)\, g$

(Section 4.6.1)

= 0.42 x (7,03,000 + 15,902 + 5,097) x 9.81

= 2,699 kN

Similarly, base shear in convective mode,

$V_c = (A_h)_c\, m_c\, g$ (Section 4.6.1)

= 0.05 x 3,09,000 x 9.81

= 152 kN

Total base shear at the bottom of wall,

$V = \sqrt{V_i^2 + V_c^2}$ (Section 4.6.3)

$= \sqrt{(2{,}699)^2 + (152)^2}$

= 2,703 kN.

Total lateral base shear is about 27 % of seismic weight (10,016 kN) of tank.

4.6. Moment at Bottom of Wall

Bending moment at the bottom of wall in impulsive mode,

$M_i = (A_h)_i [\, m_i h_i + m_w h_w + m_t h_t\,]\, g$

(Section 4.7.1.1)

= 0.38 x [(7,03,000 x 3.32) + (15,902 x 5.25) + (5,097 x 10.5025)] x 9.81

= 9,211 kN-m

Similarly, bending moment in convective mode,

$M_c = (A_h)_c\, m_c\, h_c\, g$

(Section 4.7.1.1)

= 0.05 x 3,09,000 x 5.98 x 9.81

= 906 kN-m

Total bending moment at bottom of wall,

$M = \sqrt{M_i^{\,2} + M_c^{\,2}}$ (Section 4.7.3)

$= \sqrt{(9{,}211)^2 + (906)^2}$

= 9,255 kN-m.

4.7. Overturning Moment

Overturning moment at the bottom of base plate in impulsive mode,

$M_i^* = (A_h)_i [\, m_i (h_i^* + t_b) + m_w (h_w + t_b) + m_t (h_t + t_b) + m_b t_b / 2]\, g$

(Section 4.7.1.2)

= 0.38 x [(7, 03,000 x (5.19 + 0.01)) + (15,902 x (5.25 + 0.01) + (5,097 x (10.5025 + 0.01)) + (9,072 x 0.01 / 2)] x 9.81

= 14,139 kN-m.

Similarly, overturning moment in convective mode,

$M_c^* = (A_h)_c\, m_c (\, h_c^* + t_b)\, g$

(Section 4.7.1.2)

= 0.05 x 3,09,000 x (6.43 + 0.01) x 9.81

= 976 kN-m.

Total overturning moment at the bottom of base plate,

$M^* = \sqrt{M_i^{*2} + M_c^{*2}}$ (Section 4.7.3)

$= \sqrt{(14{,}139)^2 + (976)^2}$

= 14,173 kN-m.

4.8. Hydrodynamic Pressure

4.8.1. Impulsive Hydrodynamic Pressure

Impulsive hydrodynamic pressure on wall

$p_{iw}(y) = Q_{iw}(y)\, (A_h)_i\, \rho\, g\, h \cos\phi$

$Q_{iw}(y) = 0.866\, [1 - (y / h)^2\,] \tanh(0.866\, D / h)$

(Section 4.9.1(a))

Maximum pressure will occur at $\phi = 0$.

At base of wall, $y = 0$;

$Q_{iw}(y = 0)$

$= 0.866[1 - (0 / 8.84)^2] \times \tanh(0.866 \times 12 / 8.84)$

= 0.72.

Impulsive pressure at the base of wall,

$p_{iw}(y = 0)$ = 0.72 x 0.38 x 1,000 x 9.81 x 8.84 x 1

= 23.73 kN/m^2.

Impulsive hydrodynamic pressure on the base slab ($y = 0$)

$$p_{ib} = 0.866 (A_h)_i \rho g h \sinh(0.866x/L)/\cosh(0.866 l' / h)$$

(Section 4.9.1(a))

$= 0.866 \times 0.38 \times 1{,}000 \times 9.81 \times 8.84 \times \sinh\ (0.866 \times 12\ /\ (2 \times 8.84))\ /\ \cosh\ (0.866 \times 12 / 2 \times 8.84)$

$= 15.07$ kN/m^2

4.8.2. Convective Hydrodynamic Pressure

Convective hydrodynamic pressure on wall,

$p_{cw} = Q_{cw}(y)\ (A_h)_c\ \rho\, g\, D\ [1- 1/3 \cos^2 \phi] \cos\phi$

$Q_{cw}(y) = 0.5625 \cosh(3.674y/D)/ \cosh(3.674h /D)$

(Section 4.9.2(a))

Maximum pressure will occur at $\phi = 0$.

At base of wall, $y = 0$;

$Q_{cw}(y = 0) = 0.5625 \times \cosh\ (0 / 12) / \cosh\ (3.674 \times 8.84 /12)$

$= 0.07$.

Convective pressure at the base of wall,

$p_{cw}(y = 0) = 0.07 \times 0.05 \times 1{,}000 \times 9.81 \times 12 \times 0.67 \times 1$

$= 0.28$ kN/m^2

At $y = h$;

$Q_{cw}(y = h) = 0.5625$

Convective pressure at $y = h$,

$p_{cw}(y = h)$

$= 0.5625 \times 0.05 \times 1{,}000 \times 9.81 \times 12 \times 0.67 \times 1$

$= 2.22$ kN/m^2.

Convective hydrodynamic pressure on the base slab ($y = 0$)

$p_{cb} = Q_{cb}(x)\ (A_h)_c\ \rho\ g\ D$

$Q_{cb}(x) = 1.125[x/D – 4/3\ (x/D)^3] \text{ sech } (3.674\ h/D)$

(Section 4.9.2(a))

$= 1.125[D/2D – 4/3\ (D/2D)^3] \text{ sech } (3.674 \times 8.84 /12)$

$= 0.05$

Convective pressure on top of base slab ($y = 0$)

$p_{cb} = 0.05 \times 0.05 \times 1{,}000 \times 9.81 \times 12$

$= 0.30$ kN/m^2

4.8.3. Equivalent Linear Pressure Distribution

For stress analysis of tank wall, it is convenient to have linear pressure distribution along wall height. As per Section 4.9.4, equivalent linear distribution for impulsive hydrodynamic pressure distribution will be as follows:

Base shear due to impulsive liquid mass per unit circumferential length,

$$q_i = \frac{(A_h)_i m_i g}{\pi D/2} = \frac{0.38 \times 7{,}03{,}000 \times 9.81}{\pi \times 12/2}$$

$= 139.0$ kN/m

Pressure at bottom and top is given by,

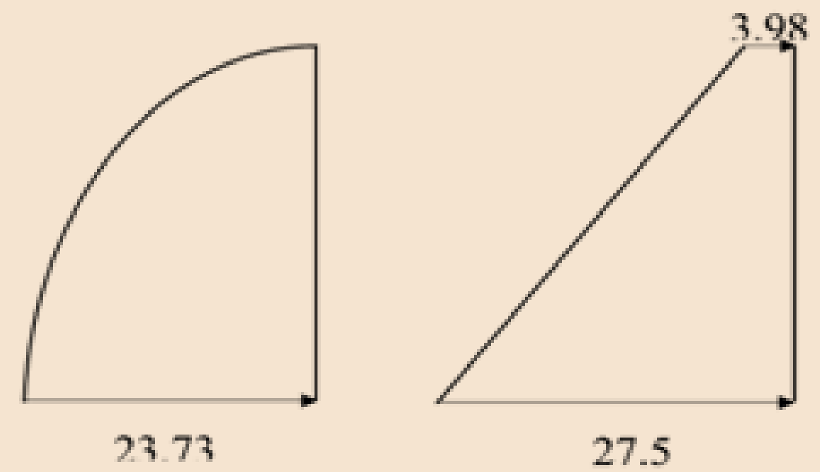
$$a_i = \frac{q_i}{h^2}(4h - 6h_i) = \frac{139.0}{8.84^2}(4 \times 8.84 - 6 \times 3.32)$$

$= 27.5$ kN/m^2

$$b_i = \frac{q_i}{h^2}(6h_i - 2h) = \frac{139.0}{8.84^2}(6 \times 3.32 - 2 \times 8.84)$$

$= 3.98$ kN/m^2

Equivalent linear impulsive pressure distribution is shown below:



Actual distribution

Linearised distribution

Similarly, equivalent linear distribution for convective pressure can be obtained as follows:

Base shear due to convective liquid mass per unit circumferential length, q_c

$$q_c = \frac{(A_h)_c m_c g}{\pi D/2} = \frac{0.05 \times 3{,}09{,}000 \times 9.81}{\pi \times 12/2}$$

$= 8.04$ kN/m

Pressure at bottom and top is given by,

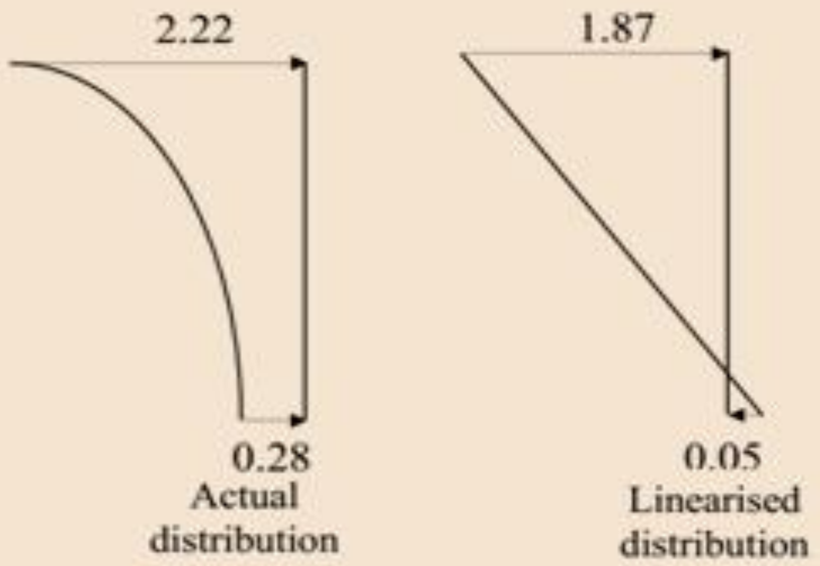
$$a_c = \frac{q_c}{h^2}(4h - 6h_c) = \frac{8.04}{8.84^2}(4 \times 8.84 - 6 \times 5.98)$$

$= - 0.05$ kN/m^2

$$b_c = \frac{q_c}{h^2}(6h_c - 2h) = \frac{8.04}{8.84^2}(6 \times 5.98 - 2 \times 8.84)$$

$$= 1.87 \text{ kN/m}^2$$

Equivalent linear convective pressure distribution is shown below:



It may be noted that the linearised distribution for convective pressure has a very small negative value at the base. For design purpose this may be taken as zero.

4.9. Pressure Due to Wall Inertia

Pressure on wall due to its inertia,

$p_{ww} = (A_h)_i \, t \, \rho_m \, g$ (Section 4.9.5)

$= 0.38 \times 0.005 \times 78.53$

$= 0.15 \text{ kN / m}^2$

This pressure is uniformly distributed along the wall height.

It may be noted that for this steel tank pressure due to wall inertia is negligible compared to impulsive hydrodynamic pressure.

4.10. Pressure Due to Vertical Excitation

Hydrodynamic pressure on tank wall due to vertical ground acceleration,

$p_v = (A_v)\,[\rho \, g \, h\,(1 - y / h)]$

(Section 4.10.1)

$$(A_v) = \frac{2}{3}\left(\frac{Z}{2}\frac{I}{R}\frac{S_a}{g}\right)$$

$Z = 0.36$ (IS 1893(Part 1): Table 2; Zone V)

$I = 1.5$ (Table 1)

$R = 2.5$

Since time period of vertical mode of vibration is recommended as 0.3 sec in Section 4.10.1, for 2 % damping,

$S_a/g = 2.5 \times 1.4 = 3.5$

Hence,

$$(A_v) = \frac{2}{3}\left(\frac{Z}{2}\frac{I}{R}\frac{S_a}{g}\right)$$

$$= \frac{2}{3} \times \left(\frac{0.36}{2} \times \frac{1.5}{2.5} \times 3.5\right)$$

$$= 0.25$$

At the base of wall, i.e., $y = 0$,

$p_v = 0.25 \times [1{,}000 \times 9.81 \times 8.84 \times (1 - 0 / 8.84)]$

$= 21.7 \text{ kN/m}^2$

4.11. Maximum Hydrodynamic Pressure

Maximum hydrodynamic pressure,

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}^{\,2} + p_v^{\,2}}$$

(Section 4.10.2)

At the base of wall,

$$p = \sqrt{(23.73 + 0.15)^2 + 0.28^2 + 21.7^2}$$

$= 32.3 \text{ kN/m}^2$.

Maximum hydrodynamic pressure is about 37% of hydrostatic pressure ($\rho \, g \, h = 1{,}000 \times 9.81 \times 8.84 = 86.72 \text{ kN/m}^2$). Hence, hydrodynamic pressure will marginally influence container design, as permissible stresses are already increased by 33%.

4.12. Sloshing Wave Height

Maximum sloshing wave height,

$d_{max} = (A_h)_c \, R \, D / 2$ (Section 4.11)

$= 0.05 \times 2.5 \times 12 / 2$

$= 0.75 \text{ m}$

4.13. Anchorage Requirement

Here, $\frac{h}{D} = \frac{8.84}{12} = 0.74$;

$$\frac{1}{(A_h)_i} = \frac{1}{0.38} = 2.63$$

As $\frac{h}{D} < \frac{1}{(A_h)_i}$

No anchorage is required.

(Section 4.12)

Example 5 – Ground SupportedCircular Concrete Tank

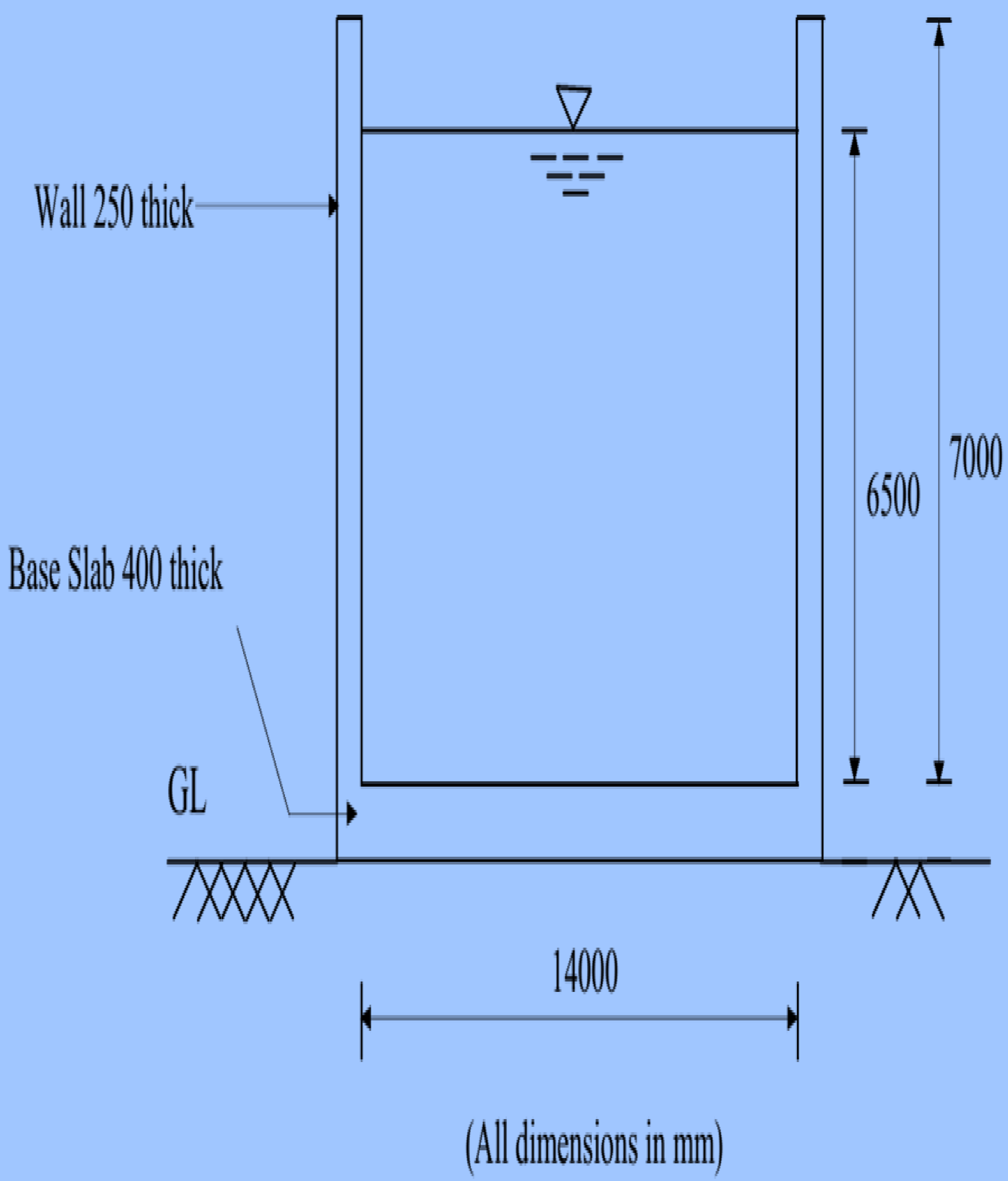
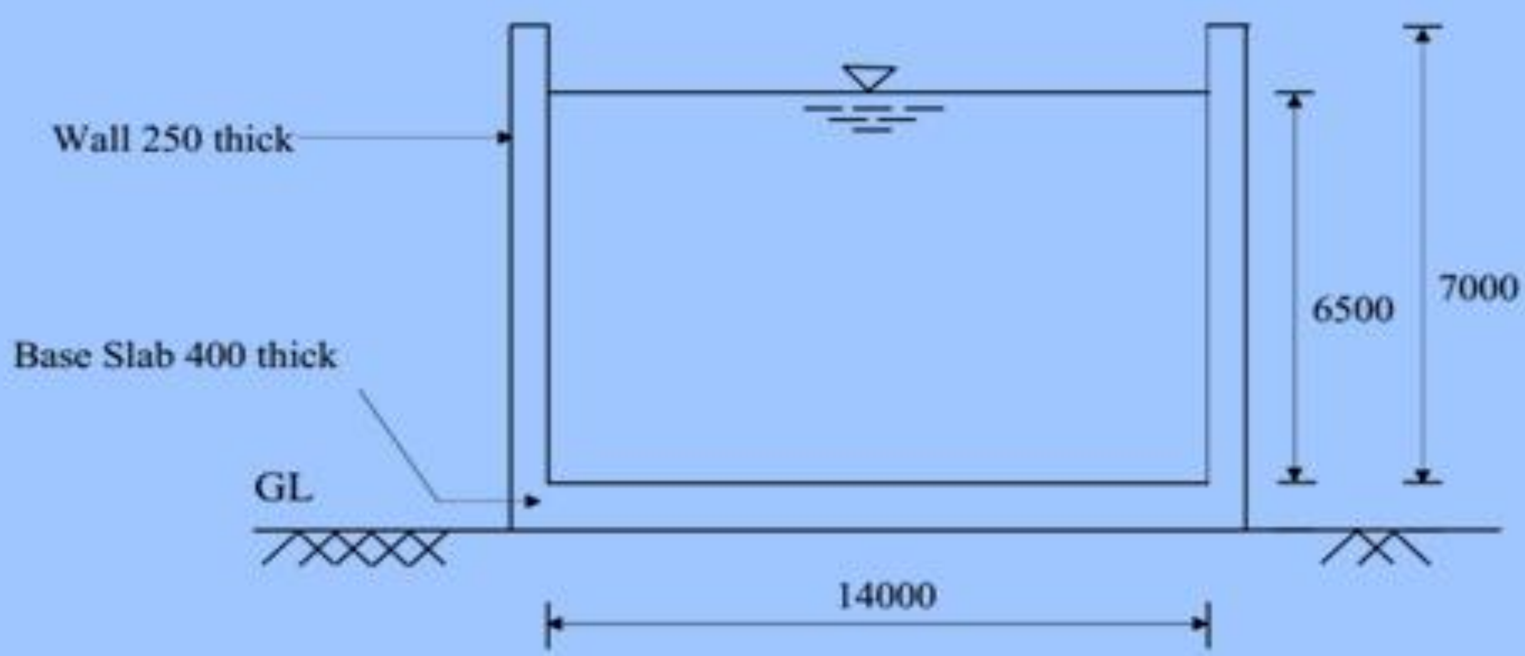


Figure 5.1 Sectional elevation

Example 5 – Ground Supported Circular Concrete Tank

5. Problem Statement:

A ground supported cylindrical RC water tank without roof has capacity of 1,000 m^3. Inside diameter of tank is 14 m and height is 7.0 m (including a free board of 0.5 m). Tank wall has uniform thickness of 250 mm and base slab is 400 mm thick. Grade of concrete is M30. Tank is located on soft soil in seismic zone IV. Density of concrete is 25 kN/m^3. Analyze the tank for seismic loads.



(All dimensions in mm)

Figure 5.1 Sectional elevation

Solution:

5.1. Weight Calculations

Weight of tank wall

$= \pi \times (14 + 0.25) \times 0.25 \times 25 \times 7.0$

$= 1{,}959$ kN

Mass of tank wall, m_w

$= 1{,}959 \times 1{,}000 / 9.81$

$= 1{,}99{,}694$ kg

Mass of base slab, m_b

$= \pi \times (7.25)^2 \times 0.4 \times 25 \times 1{,}000 / 9.81$

$= 1{,}68{,}328$ kg.

Volume of water = 1,000 m^3

Mass of water, m = 10,00,000 kg

Weight of water = 9,810 kN

5.2. Parameters of Spring Mass Model

$h = 6.5$ m; $D = 14$ m

For $h / D = 6.5/14 = 0.46$,

$m_i / m = 0.511$;

$m_i = 0.511 \times 10{,}00{,}000 = 5{,}11{,}000$ kg

$m_c / m = 0.464$;

$m_c = 0.464 \times 10{,}00{,}000 = 4{,}64{,}000$ kg

$h_i / h = 0.375$; $\quad h_i = 0.375 \times 6.5 = 2.44$ m

$h_c / h = 0.593$; $\quad h_c = 0.593 \times 6.5 = 3.86$ m

$h_i^* / h = 0.853$; $\quad h_i^* = 0.853 \times 6.5 = 5.55$ m

$h_c^* / h = 0.82$; $\quad h_c^* = 0.82 \times 6.5 = 5.33$ m

(Section 4.2.1.2)

Note that about 51% of liquid is excited in impulsive mode while 46% participates in convective mode. Sum of impulsive and convective mass is about 2.5 % less than mass of liquid.

5.3. Time Period

Time period of impulsive mode,

$$T_i = \frac{C_i h \sqrt{\rho}}{\sqrt{(t / D)}\sqrt{E}}$$

Where,

h = Depth of liquid = 6.5 m,

ρ = Mass density of water = 1,000 kg/m^3,

t = Thickness of wall = 0.25 m,

D = Inner diameter of tank = 14 m ,

E = Young's modulus

$= 5{,}000\ \sqrt{f_{ck}}$

$= 5{,}000 \times \sqrt{30}$

= 27,390 N/mm^2

= 27,390 x 10^6 N/m^2.

For $h / D = 0.46$, $C_i = 4.38$

(Section 4.3.1.1)

$$T_i = \frac{4.38 \times 6.5 \times \sqrt{1{,}000}}{\sqrt{(0.25/14)} \times \sqrt{27{,}390 \times 10^6}}$$

$= 0.04$ sec.

Time period of convective mode,

$$T_c = C_c\sqrt{\frac{D}{g}}$$

For $h / D = 0.46$, $C_c = 3.38$

(Section 4.3.1.1)

$$T_c = 3.38\sqrt{\frac{14}{9.81}} = 4.04 \text{ sec.}$$

5.4. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

This tank has fixed base hence R is taken as 2.0.

(Table 2)

Here, $T_i = 0.04$ sec,

Site has soft soil,

Damping = 5%, (Section 4.4)

Since $T_i < 0.1$ sec as per Section 4.5.2,

$(S_a/g)_i = 2.5$

$$(A_h)_i = \frac{0.24}{2} \times \frac{1.5}{2.0} \times 2.5 = 0.225$$

Design horizontal seismic coefficient for convective mode,

$$(A_h)_c = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_c$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.24$ (IS 1893(Part 1): Table 2; Zone IV)

$I = 1.5$ (Table 1)

For convective mode, value of R is taken same as that for impulsive mode as per Section 4.5.1.

Here, $T_c = 4.04$ sec,

Site has soft soil,

Damping = 0.5%, (Section 4.4)

Hence, as per Section 4.5.3 and IS 1893(Part 1): 2002, Figure 2

$(S_a/g)_c = 1.75 \times 0.413 = 0.72$

Multiplying factor of 1.75 is used to obtain S_a/g values for 0.5% damping from that for 5% damping.

(Section 4.5.4)

$$(A_h)_c = \frac{0.24}{2} \times \frac{1.5}{2.0} \times 0.72 = 0.065$$

5.5. Base Shear

Base shear at the bottom of wall in impulsive mode,

$V_i = (A_h)_i (m_i + m_w + m_t)\, g$

(Section 4.6.1)

= 0.225 x (5,11,000 + 1,99,694 + 0) x 9.81

= 1,569 kN

Similarly, base shear in convective mode,

$V_c = (A_h)_c\, m_c\, g$ (Section 4.6.1)

= 0.065 x 4,64,000 x 9.81

= 296 kN

Total base shear at the bottom of wall,

$V = \sqrt{V_i^2 + V_c^2}$ (Section 4.6.3)

$= \sqrt{(1{,}569)^2 + (296)^2}$

= 1,597 kN.

Total lateral base shear is about 14 % of seismic weight (11,769 kN) of tank.

5.6. Moment at Bottom of Wall

Bending moment at the bottom of wall in impulsive mode,

$M_i = (A_h)_i [\, m_i h_i + m_w h_w + m_t h_t \,]\, g$

(Section 4.7.1.1)

= 0.225 x [(5,11,000 x 2.44)

+ (1,99,694 x 3.5) + 0] x 9.81

= 4,295 kN-m

Similarly, bending moment in convective mode,

$M_c = (A_h)_c\, m_c h_c\, g$

(Section 4.7.1.1)

= 0.065 x 4,64,000 x 3.86 x 9.81

= 1,142 kN-m

Total bending moment at bottom of wall,

$M = \sqrt{M_i^2 + M_c^2}$ (Section 4.7.3)

$= \sqrt{(4{,}295)^2 + (1{,}142)^2}$

= 4,444 kN-m.

5.7. Overturning Moment

Overturning moment at the bottom of base slab in impulsive mode,

$M_i^* = (A_h)_i [\, m_i (h_i^* + t_b) + m_w (h_w + t_b) + m_t (h_t + t_b) + m_b t_b / 2 \,]\, g$

(Section 4.7.1.2)

=0.225x[(5,11,000x(5.55 + 0.4) + (1,99,694 x (3.5+ 0.4)) + 0 + (1,68,328 x 0.4 / 2)] x 9.81

= 8,504 kN-m.

Similarly, overturning moment in convective mode,

$M_c^* = (A_h)_c\, m_c\, (h_c^* + t_b)\, g$

(Section 4.7.1.2)

= 0.065 x 4,64,000 x (5.33 + 0.4) x 9.81

= 1,695 kN-m.

Total overturning moment at the bottom of base slab,

$M^* = \sqrt{M_i^{*2} + M_c^{*2}}$ (Section 4.7.3)

$= \sqrt{(8{,}504)^2 + (1{,}695)^2}$

= 8,671 kN-m.

5.8. Sloshing Wave Height

Maximum sloshing wave height,

$d_{max} = (A_h)_c\, R\, D / 2$ (Section 4.11)

= 0.065 x 2.0 x 14 / 2

= 0.91 m

Sloshing wave height exceeds the freeboard of 0.5 m.

5.9. Anchorage Requirement

Here, $\frac{h}{D} = \frac{6.5}{14} = 0.46$;

$\frac{1}{(A_h)_i} = \frac{1}{0.225} = 4.4$

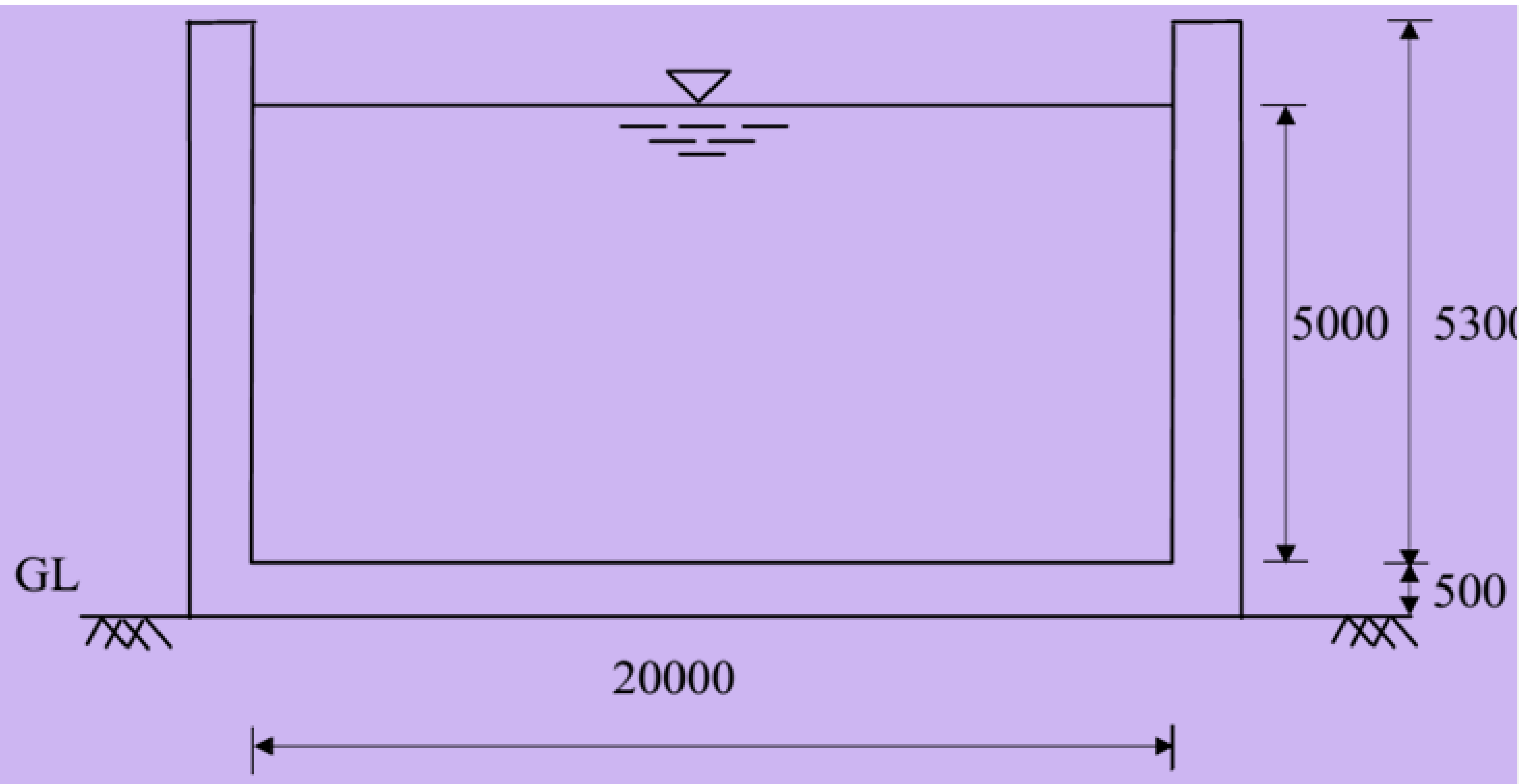
As $\frac{h}{D} < \frac{1}{(A_h)_i}$

No anchorage is required.

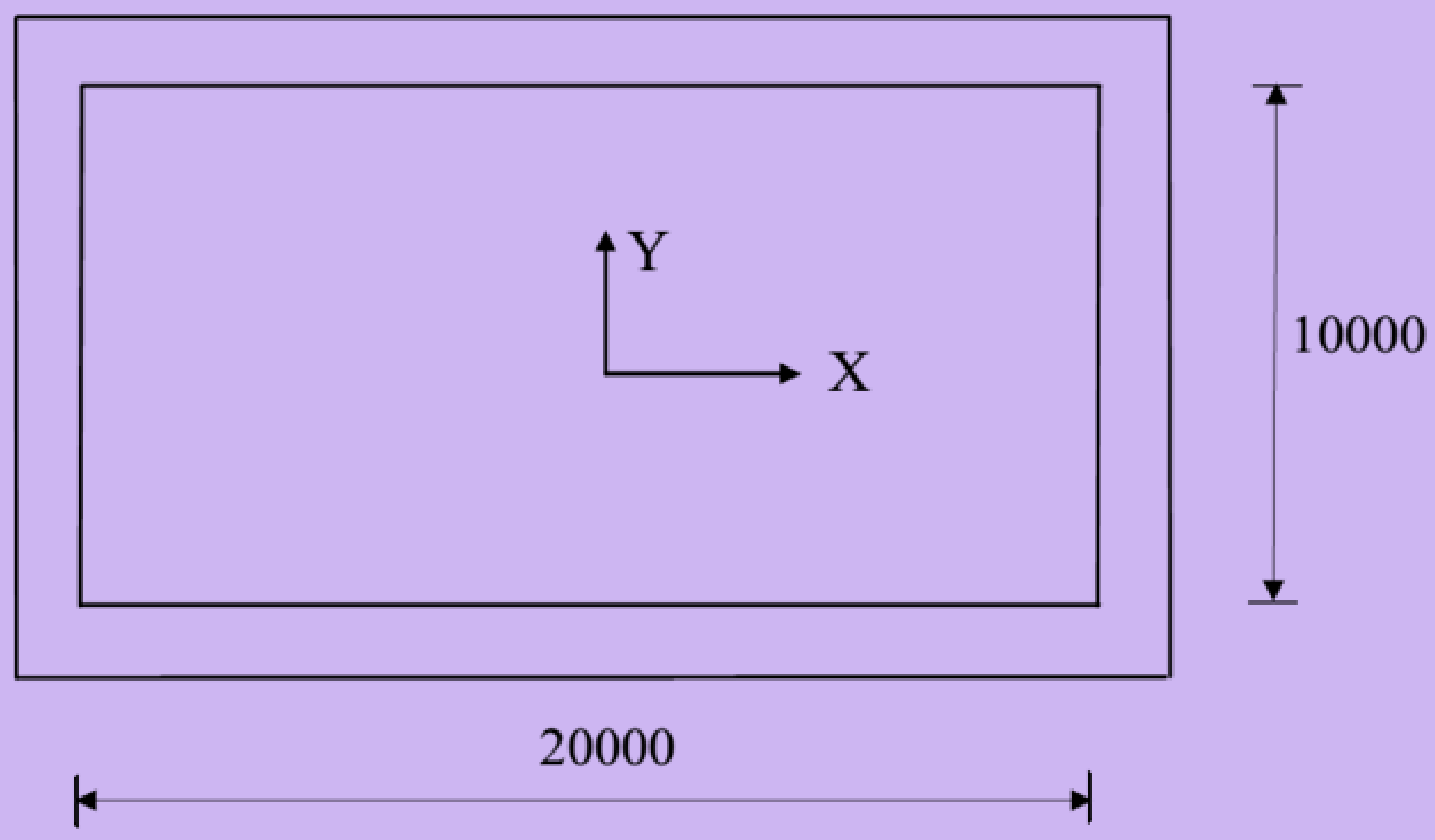
(Section 4.12)

Hydrodynamic pressure calculations for this tank are not shown. These will be similar to those in Example 4.

Example 6: Ground Supported Rectangular Concrete Tank



(a) Elevation



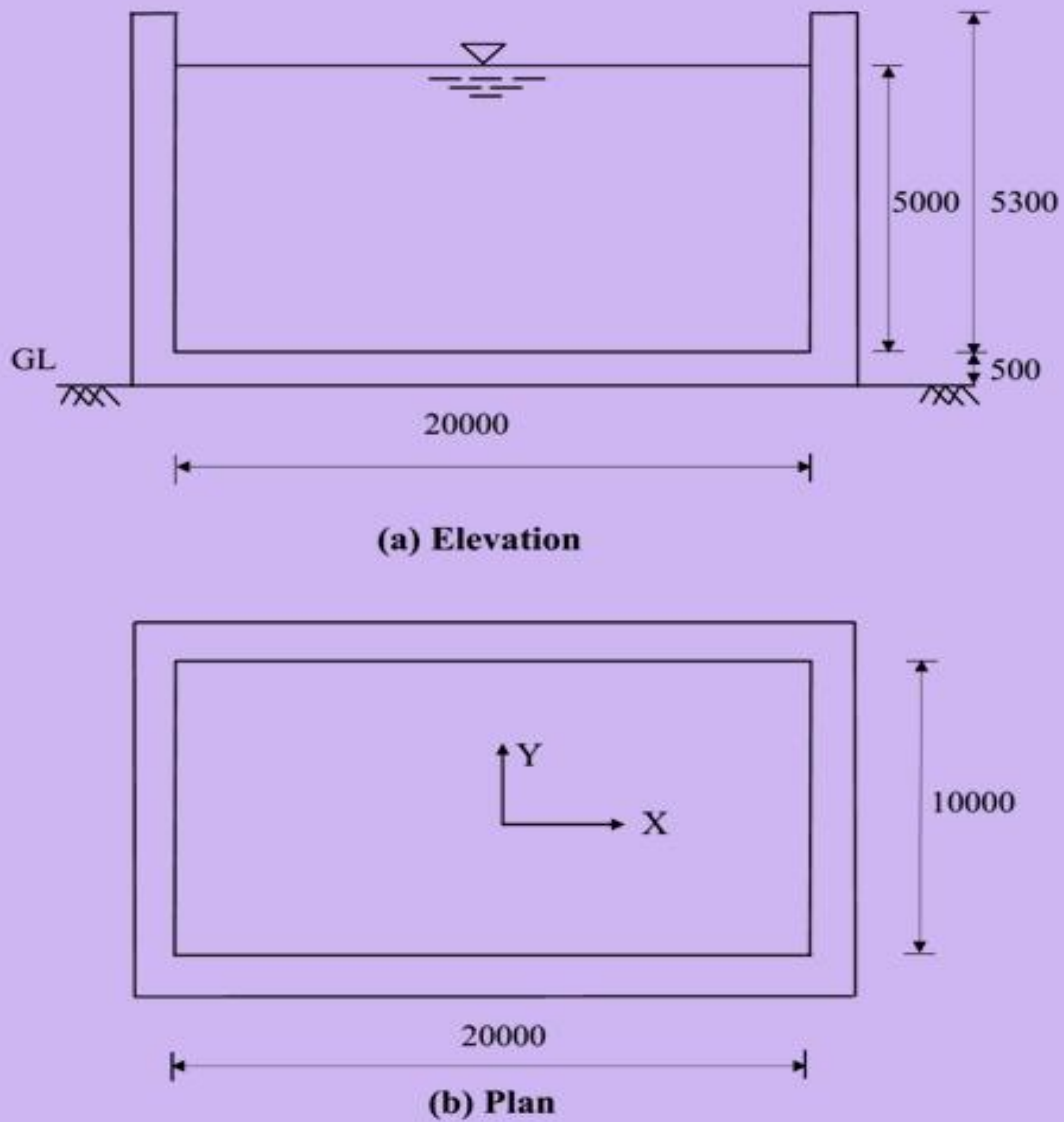
(b) Plan

(All Dimensions in mm)

Figure 6.1 Details of tank geometry

6. Problem Statement:

A ground supported rectangular RC water tank of 1,000 m^3 capacity has plan dimensions of 20 x 10 m and height of 5.3 m (including a free board of 0.3 m). Wall has a uniform thickness of 400 mm. The base slab is 500 mm thick. There is no roof slab on the tank. Tank is located on hard soil in Zone V. Grade of concrete is M30.Analyze the tank for seismic loads.



(All Dimensions in mm)

Figure 6.1 Details of tank geometry

6.1. Weight Calculations

Weight of tank wall

= 2 x (20.4 + 10.4) x 0.4 x 25 x 5.3

= 3,265 kN

Mass of tank wall, m_w

= 3,265 x 1,000 / 9.81

= 3,32,824 kg.

Mass of base slab, m_b

= 10.8 x 20.8 x 0.5 x 25 x 1,000 / 9.81

= 2,86,239 kg.

Volume of water = 1,000 m^3

Weight of water = 10 x 20 x 5 x 9.81 = 9,810 kN

Mass of water, m = 10,00,000 kg

For rectangular tank, seismic analysis is to be performed for loading in X- and Y- directions.

6.2. Analysis along X-Direction

This implies that earthquake force is applied in X-direction. For this case, L = 20 m and B = 10 m.

6.2.1. Parameters of Spring Mass Model

For h / L = 5/20 = 0.25,

m_i / m = 0.288;

m_i = 0.288 x 10,00,000 = 2,88,000 kg

m_c / m = 0.695;

m_c = 0.695 x 10,00,000 = 6,95,000 kg

h_i / h = 0.375 ; h_i = 0.375 x 5 = 1.88 m

h_c / h = 0.524 ; h_c = 0.524 x 5 = 2.62 m

h_i^* / h = 1.61 ; h_i^* = 1.61 x 5 = 8.05 m

h_c^* / h = 2.0 ; h_c^* = 2.0 x 5 = 10.0 m.

(Section 4.2.1.2)

For this case, h/L = 0.25, i.e. tank is quite squat and hence, substantial amount of mass (about 70%) participates in convective mode; and about 30% liquid mass contributes to impulsive mode. Sum total of convective and impulsive mass is about 1.7% less than total liquid mass.

6.2.2. Time Period

Time period of impulsive mode,

$$T_i = 2\pi\sqrt{\frac{d}{g}}$$ (Section 4.3.1.2)

Where, d = deflection of the tank wall on the vertical center-line at a height $\bar{h}$ when loaded by a uniformly distributed pressure q,

(Section 4.3.1.2)

$$\bar{h} = \frac{\frac{m_i}{2}h_i + \bar{m}_w \frac{h}{2}}{\frac{m_i}{2} + \bar{m}_w}$$

$\bar{m}_w$ = mass of one tank wall perpendicular to direction of loading.

Mass of one wall is obtained by considering its inner dimensions only.

= 5.3 x 0.4 x 10 x 25 x 1,000 / 9.81

= 54,027 kg

Hence,

$$\bar{h} = \frac{\frac{2,88,000}{2} \times 1.88 + 54,027 \times \frac{5.3}{2}}{\frac{2,88,000}{2} + 54,027} = 2.09 \text{ m}$$

$$q = \frac{\left(\frac{m_i}{2} + \bar{m}_w\right)g}{B\ h}$$

$$= \frac{\left(\frac{2,88,000}{2} + 54,027\right) \times 9.81}{10 \times 5}$$

$= 38.9$ kN/m^2

To find the deflection of wall due to this pressure, it can be considered to be fixed at three edges and free at top.

Deflection of wall can be obtained by performing analysis of wall or by classical analysis using theory of plates. However, here, simple approach given in commentary of Section 4.3.1.2 is followed. As per this approach a strip of unit width of wall is considered as a cantilever and subjected to a concentrated force $P = q$ x h x 1 = 38.9 x 5 x 1 = 194.5 kN. Length of the cantilever is $\bar{h}$. Hence,

$$d = \frac{P(\bar{h})^3}{3\,E\,I_w}$$

Where,

$E = 5,000\sqrt{f_{ck}}$ = 5,000 x $\sqrt{30}$

= 27,390 N/mm^2

= 27.39 x10^6 kN/m^2

I_w = Moment of inertia of cantilever

$= 1.0 \text{ x } \frac{t^3}{12} = 1.0 \times \frac{0.4^3}{12} = 5.33 \times 10^{-3}\, m^4$

Hence,

$$d = \frac{194.5 \times 2.09^3}{3 \times 27.39 \times 10^6 \times 5.33 \times 10^{-3}} = 0.00405\, m$$

$$T_i = 2\pi\sqrt{\frac{0.00405}{9.81}} = 0.13 \sec.$$

Time period of convective mode,

$$T_c = C_c\sqrt{\frac{L}{g}}$$

For $h / L = 0.25$, $C_c = 4.36$

(Section 4.3.2.2(b))

$$T_c = 4.36 \times \sqrt{\frac{20}{9.81}} = 6.22 \text{ sec.}$$

6.2.3. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Section 4.5.1)

Where,

$Z = 0.36$ (IS 1893(Part 1): Table 2; Zone V)

$I = 1.5$ (Table 1)

Since this RC tank is fixed at base, R is taken as 2.0. (Table 2)

Here, $T_i = 0.13$ sec,

Site has hard soil,

Damping = 5%, (Section 4.4)

Hence, $(S_a/g)_i = 2.5$

(IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.36}{2} \times \frac{1.5}{2.0} \times 2.5 = 0.34$$

Design horizontal seismic coefficient for convective mode,

$$(A_h)_c = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_c$$

(Sections 4.5 and 4.5.1)

Where,

$Z = 0.36$ (IS 1893(Part 1): Table 2; Zone V)

$I = 1.5$ (Table 1)

For convective mode, value of R is taken same as that for impulsive mode as per Section 4.5.1.

Here, $T_c = 6.22$ sec,

Site has hard soil,

Damping = 0.5%, (Section 4.4)

Hence, as per Section 4.5.3 and IS 1893(Part 1): 2002, Figure 2

$(S_a/g)_c = 1.75 \times 0.16 = 0.28$

Multiplying factor of 1.75 is used to obtain S_a/g values for 0.5 % damping from that for 5 % damping.

(Section 4.5.4)

$$(A_h)_c = \frac{0.36}{2} \times \frac{1.5}{2.0} \times 0.28 = 0.038$$

6.2.4. Base Shear

Base shear at the bottom of wall in impulsive mode,

$V_i = (A_h)_i\,(m_i + m_w + m_t)g$

(Section 4.6.1)

$= 0.34 \times (2{,}88{,}000 + 3{,}32{,}824 + 0) \times 9.81$

$= 2{,}071$ kN.

Similarly, base shear in convective mode,

$V_c = (A_h)_c\, m_c\, g$ (Section 4.6.1)

$= 0.038 \times 6{,}95{,}000 \times 9.81$

$= 259$ kN

Total base shear at the bottom of wall,

$V = \sqrt{V_i^2 + V_c^2}$ (Section 4.6.3)

$= \sqrt{(2{,}071)^2 + (259)^2}$

$= 2{,}087$ kN.

This lateral base shear is about 16 % of total seismic weight (13,075 kN) of tank.

6.2.5. Moment at Bottom of Wall

Bending moment at the bottom of wall in impulsive mode,

$M_i = (A_h)_i\,[\, m_i h_i + m_w h_w + m_t h_t\,]\, g$

(Section 4.7.1.1)

$= 0.34 \times [(2{,}88{,}000 \times 1.88) +$

$(3{,}32{,}824 \times 2.65) + 0] \times 9.81$

$= 4{,}747$ kN-m

Similarly, bending moment in convective mode,

$M_c = (A_h)_c\, m_c\, h_c\, g$

(Section 4.7.1.1)

$= 0.038 \times 6{,}95{,}000 \times 2.62 \times 9.81$

$= 679$ kN-m

Total bending moment at bottom of wall,

$M = \sqrt{M_i^2 + M_c^2}$ (Section 4.7.3)

$= \sqrt{(4{,}747)^2 + (679)^2}$

= 4,795 kN-m.

6.2.6. Overturning Moment

Overturning moment at the bottom of base slab in impulsive mode,

$M_i^* = (A_h)_i\,[\,m_i\,(h_i^* + t_b) + m_w\,(h_w + t_b) + m_t\,(h_t + t_b) + m_b\,t_b/2]\,g$

(Section 4.7.1.2)

= 0.34 x [(2,88,000 x (8.05 + 0.5) +

(3,32,824 x (2.65 + 0.5) + 0

+ (2,86,239 x 0.5 / 2)] x 9.81

= 11,948 kN-m.

Similarly, overturning moment in convective mode,

$M_c^* = (A_h)_c\ m_c\,(\,h_c^* + t_b\,)\,g$

(Section 4.7.1.2)

= 0.038 x 6,95,000 x (10 + 0.5) x 9.81

= 2,721 kN-m.

Total overturning moment at the bottom of base slab,

$M^* = \sqrt{M_i^{*2} + M_c^{*2}}$ (Section 4.7.3)

$= \sqrt{(11{,}948)^2 + (2{,}721)^2}$

= 12,254 kN-m.

6.2.7. Hydrodynamic Pressure

6.2.7.1. Impulsive Hydrodynamic Pressure

Impulsive hydrodynamic pressure on wall is

$p_{iw} = Q_{iw}(y)\ (A_h)_i\ \rho\ g\ h$

$Q_{iw}(y) = 0.866[1\text{-}(y/h)^2]$ x tanh (0.866 L/h)

(Section 4.9.1.(b))

At base of wall, $y = 0$;

$Q_{iw}(y = 0\,) = 0.866\ [1\text{-}(0/5)^2]$ x tanh(0.866 x 20/5)

= 0.86.

Impulsive pressure at the base of wall,

$p_{iw}(\,y = 0\,)$ = 0.86 x 0.34 x 1,000 x 9.81 x 5

= 14.3 kN/m^2.

Impulsive hydrodynamic pressure on the base slab ($y = 0$)

$p_{ib} = Q_{ib}(x)\ (A_h)_i\ \rho\ g\ h$

$Q_{ib}(x) = \sinh(0.866\ x/L)/\cosh(0.866\ L/h)$

(Section 4.9.1(a))

= sinh (0.866 x 20 /10) /cosh (0.866 x 20/5)

= 0.171

Impulsive pressure on top of base slab ($y = 0$)

p_{ib} = 0.171 x 0.34 x 1,000 x 9.81 x 5

= 2.9 kN/m^2

6.2.7.2. Convective Hydrodynamic Pressure

Convective hydrodynamic pressure on wall is

$p_{cw} = Q_{cw}(y)\ (A_h)_c\ \ \rho\ g\ L$

$$Q_{cw}(y) = 0.4165\,\frac{\cosh\left(3.162\,\frac{y}{L}\right)}{\cosh\left(3.162\,\frac{h}{L}\right)}$$

(Section 4.9.2.(b))

At base of wall, $y = 0$;

$$Q_{cw}(\,y = 0\,) = 0.4165 \times \frac{\cosh\left(3.162 \times \frac{0}{20}\right)}{\cosh\left(3.162 \times \frac{5}{20}\right)}$$

= 0.31.

Convective pressure at the base of wall,

$p_{cw}(\,y = 0\,)$ = 0.31 x 0.038 x 1,000 x 9.81 x 20

= 2.31 kN/m^2

At $y = h$;

$Q_{cw}(\,y = h\,) = 0.4165$

Convective pressure at y = h,

$p_{cw}\,(y = h\,)$ = 0.4165 x 0.038 x 1,000 x 9.81 x 20

= 3.11 kN/ m^2

Convective hydrodynamic pressure on the base slab ($y = 0$)

$p_{cb} = Q_{cb}(x)\ (A_h)_c\ \rho\ g\ D$

$Q_{cb}(x) = 1.25[x/L - 4/3\ (x/L)^3]$ sech (3.162 h/L)

(Section 4.9.2(a))

$= 1.25[L/2L - 4/3\ (L/2L)^3]\ \text{sech}\ (3.162 \times 5/20)$

$= 0.313$

Convective pressure on top of base slab ($y = 0$)

$p_{cb} = 0.313 \times 0.038 \times 1{,}000 \times 9.81 \times 20$

$= 2.33\ \text{kN/m}^2$

6.2.8. Pressure Due to Wall Inertia

Pressure on wall due to its inertia,

$p_{ww} = (A_h)_i\, t\, \rho_m\, g$ (Section 4.9.5)

$= 0.34 \times 0.4 \times 25$

$= 3.4\ \text{kN/m}^2$.

This pressure is uniformly distributed along the wall height.

6.2.9. Pressure Due to Vertical Excitation

Hydrodynamic pressure on tank wall due to vertical ground acceleration,

$p_v = (A_v)\ [\rho\, g\, h\ (1 - y/h)]$

(Section 4.10.1)

$$(A_v) = \frac{2}{3}\left(\frac{Z}{2}\frac{I}{R}\frac{S_a}{g}\right)$$

$Z = 0.36$ (IS 1893(Part 1): Table 2; Zone V)

$I = 1.5$ (Table 1)

$R = 2.0$

Time period of vertical mode of vibration is recommended as 0.3 sec in Section 4.10, for 5% damping, $S_a/g = 2.5$.

Hence,

$$(A_v) = \frac{2}{3} \times \left(\frac{0.36}{2} \times \frac{1.5}{2.0} \times 2.5\right)$$

$= 0.225$.

At the base of wall, i.e., $y = 0$,

$p_v = 0.225 \times [1{,}000 \times 9.81 \times 5 \times (1 - 0/5)]$

$= 11.04\ \text{kN/m}^2$

6.2.10. Maximum Hydrodynamic Pressure

Maximum hydrodynamic pressure,

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}^{\ 2} + p_v^{\ 2}}$$

(Section 4.10.2)

At the base of wall,

$$p = \sqrt{(14.3 + 3.4)^2 + 2.31^2 + 11.04^2}$$

$= 21.0\ \text{kN/m}^2$.

This hydrodynamic pressure is about 43% of hydrostatic pressure ($\rho\, g\, h = 1{,}000 \times 9.81 \times 5 = 49\ \text{kN/m}^2$). In this case, hydrodynamic pressure will substantially influence the design of container.

6.2.11. Equivalent Linear Pressure Distribution

For stress analysis of tank wall, it is convenient to have linear pressure distribution along wall height. As per Section 4.9.4, equivalent linear distribution for impulsive hydrodynamic pressure distribution can be obtained as follows:

Base shear per unit circumferential length due to impulsive liquid mass,

$$q_i = \frac{(A_h)_i\, m_i\, g}{2B} = \frac{0.34 \times 2{,}88{,}000 \times 9.81}{2 \times 10}$$

$= 48.03\ \text{kN/m}$

Value of linearised pressure at bottom and top is given by,

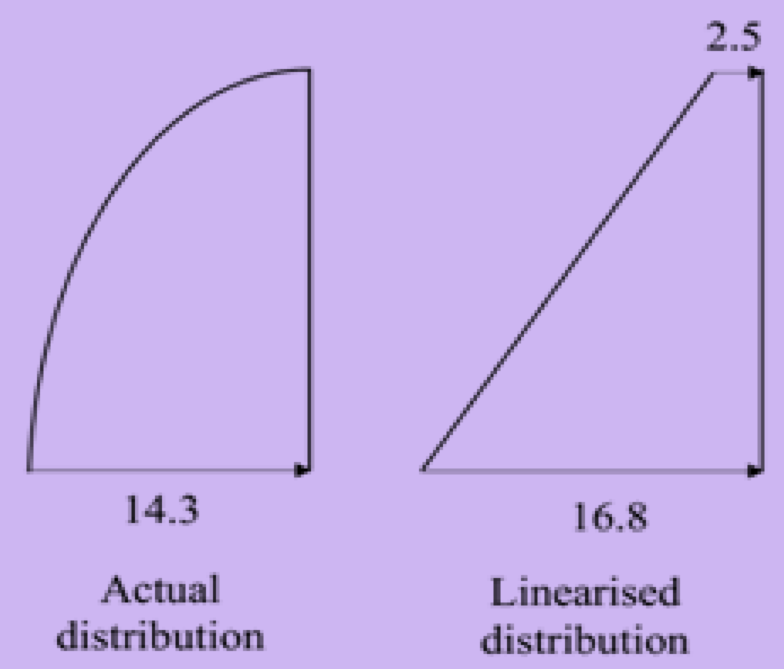
$$a_i = \frac{q_i}{h^2}(4h - 6h_i) = \frac{48.03}{5^2}(4 \times 5 - 6 \times 1.88)$$

$= 16.8\ \text{kN/m}^2$

$$b_i = \frac{q_i}{h^2}(6h_i - 2h) = \frac{48.03}{5^2}(6 \times 1.88 - 2 \times 5)$$

$= 2.5\ \text{kN/m}^2$

Equivalent linear impulsive pressure distribution is shown below:



Similarly, equivalent linear distribution for convective pressure can be obtained as follows:

Base shear due to convective liquid mass per unit circumferential length, q_c,

$$q_c = \frac{(A_h)_c m_c\, g}{2B} = \frac{0.038 \times 6{,}95{,}000 \times 9.81}{2 \times 10}$$

$$= 12.95 \text{ kN/m}$$

Pressure at bottom and top is given by,

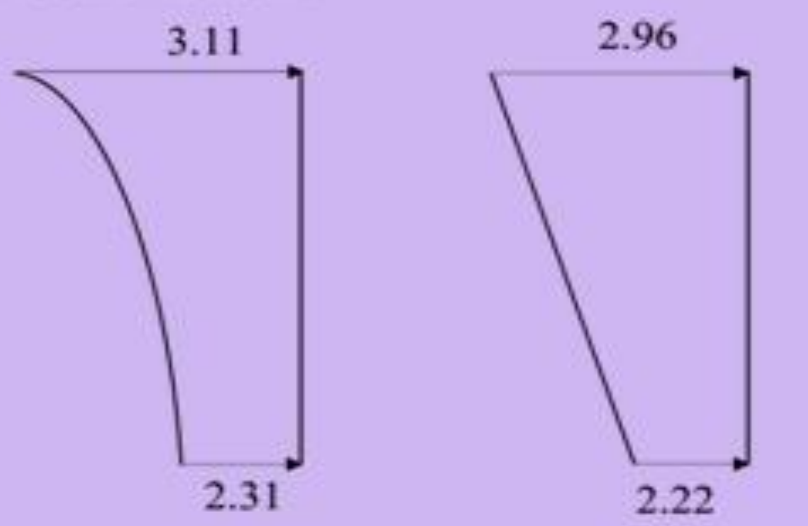
$$a_c = \frac{q_c}{h^2}(4h - 6h_c) = \frac{12.95}{5^2}(4 \times 5 - 6 \times 2.62)$$

$$= 2.22 \text{ kN/m}^2$$

$$b_c = \frac{q_c}{h^2}(6h_c - 2h) = \frac{12.95}{5^2}(6 \times 2.62 - 2 \times 5)$$

$$= 2.96 \text{ kN/m}^2$$

Equivalent linear convective pressure distribution is shown below:



Actual distribution　　Linearised distribution

6.2.12. Sloshing Wave Height

Maximum sloshing wave height,

$d_{max} = (A_h)_c\, R\, L / 2$ (Section 4.11)

$= 0.038 \times 2.0 \times 20 / 2$

$= 0.76$ m

Sloshing wave height is more than free board of 0.5 m.

6.2.13. Anchorage Requirement

Here, $\frac{h}{L} = \frac{5}{20} = 0.25$; $\frac{1}{(A_h)_i} = \frac{1}{0.34} = 2.94$

As $\frac{h}{L} < \frac{1}{(A_h)_i}$

No Anchorage is required.

(Section 4.12)

6.3. Analysis along Y-Direction

This implies that earthquake force is applied in Y-direction. For this case, L = 10 m and B = 20 m.

6.3.1. Parameters of Spring Mass Model

h/L = 5/10 = 0.50. It may be noted that for analysis in Y-direction, tank becomes comparatively less squat.

For, $h/L = 0.5$, $m_i / m = 0.542$;

$m_i = 0.542 \times 10{,}00{,}000 = 5{,}42{,}000$ kg

$m_c / m = 0.485$;

$m_c = 0.485 \times 10{,}00{,}000 = 4{,}85{,}000$ kg

$h_i / h = 0.375$; $h_i = 0.375 \times 5 = 1.88$ m

$h_c / h = 0.583$; $h_c = 0.583 \times 5 = 2.92$ m

$h_i^* / h = 0.797$; $h_i^* = 0.797 \times 5 = 4.0$ m

$h_c^* / h = 0.86$; $h_c^* = 0.86 \times 5 = 4.3$ m

(Section 4.2.1.2)

For analysis in Y-direction, liquid mass participating in convective mode is only 49% as against 70% for analysis in X-direction. This is due to change in h/L value.

6.3.2. Time Period

Time period of impulsive mode,

$$T_i = 2\pi\sqrt{\frac{d}{g}} \qquad \text{(Section 4.3.1.2)}$$

Where, d = deflection of the tank wall on the vertical center-line at a height $\bar{h}$ when loaded by a uniformly distributed pressure q,

$$\bar{h} = \frac{\frac{m_i}{2} h_i + \bar{m}_w \frac{h}{2}}{\frac{m_i}{2} + \bar{m}_w}$$

$\bar{m}_w$ = mass of one tank wall perpendicular to direction of loading.

$= 5.3 \times 0.4 \times 20 \times 25 \times 1{,}000 / 9.81$

$= 1{,}08{,}053$ kg

$$\bar{h} = \frac{\frac{5{,}42{,}000}{2} \times 1.88 + 1{,}08{,}053 \times \frac{5.3}{2}}{\frac{5{,}42{,}000}{2} + 1{,}08{,}053}$$

$= 2.1$ m.

$$q = \frac{\left(\frac{m_i}{2} + \bar{m}_w\right)g}{B\,h}$$

$$= \frac{\left(\frac{5{,}42{,}000}{2} + 1{,}08{,}053\right) \times 9.81}{20 \times 5}$$

$= 37.2$ kN/m^2

Hence, $P = 37.2 \times 1 \times 5 = 186$ kN.

As explained in Section 6.2.2 of this example,

$$d = \frac{P\left(\bar{h}\right)^3}{3EI_w}$$

Where,

$E = 27.39 \times 10^6$ kN/m^2,

$$I_w = 1.0 \times \frac{t^3}{12} = 1.0 \times \frac{0.4^3}{12} = 5.33 \times 10^{-3}\,\text{m}^4$$

$$d = \frac{186 \times 2.1^3}{3 \times 27.39 \times 10^6 \times 5.33 \times 10^{-3}} = 0.00393\,m$$

Hence, $T_i = 2\pi\sqrt{\frac{0.00393}{9.81}} = 0.13\,\text{sec}$

Time period of convective mode,

$$T_c = C_c\sqrt{\frac{L}{g}}$$

For $h / L = 0.50$, $C_c = 3.69$

(Section 4.3.2.1)

$$T_c = 3.69 \times \sqrt{\frac{10}{9.81}} = 3.73 \text{ sec}$$

6.3.3. Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient for impulsive mode,

$$(A_h)_i = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_i$$

(Section 4.5 and 4.5.1)

Where,

$Z = 0.36$ (IS 1893(Part 1): Table 2; Zone V)

$I = 1.5$ (Table 1)

Since this RC tank is fixed at base, R is taken as 2.0. (Table 2)

Here, $T_i = 0.13$ sec,

Site has hard soil,

Damping = 5%, (Section 4.4)

Hence, $(S_a/g)_i = 2.5$

(IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.36}{2} \times \frac{1.5}{2.0} \times 2.5 = 0.34$$

Design horizontal seismic coefficient for convective mode,

$$(A_h)_c = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)_c$$

(Section 4.5.1)

Where,

$Z = 0.36$ (IS 1893(Part 1): Table 2; Zone V)

$I = 1.5$ (Table 1)

For convective mode, value of R is taken same as that for impulsive mode as per Section 4.5.1.

Here, $T_c = 3.73$ sec,

Site has hard soil,

Damping = 0.5%, (Section 4.4)

Hence, as per Section 4.5.3 and IS 1893(Part 1): 2002, Figure 2

$(S_a/g)_c = 1.75 \times 0.27 = 0.47$

Multiplying factor of 1.75 is used to obtain S_a/g values for 0.5 % damping from that for 5 % damping.

(Section 4.5.4)

$$(A_h)_c = \frac{0.36}{2} \times \frac{1.5}{2.0} \times 0.47 = 0.06$$

6.3.4. Base Shear

Base shear at the bottom of wall in impulsive mode,

$V_i = (A_h)_i\,(m_i + m_w + m_t)g$

(Section 4.6.1)

$= 0.34 \times (5{,}42{,}000 + 3{,}32{,}824 + 0) \times 9.81$

$= 2{,}918$ kN

Similarly, base shear in convective mode,

$V_c = (A_h)_c\, m_c\, g$ (Section 4.6.1)

$= 0.06 \times 4{,}85{,}000 \times 9.81$

= 300 kN

Total base shear at the bottom of wall,

$$V = \sqrt{V_i^2 + V_c^2} \qquad \text{(Section 4.6.3)}$$

$$= \sqrt{(2{,}918)^2 + (300)^2}$$

= 2,933 kN.

It may be noted that total lateral base shear is about 22 % of total seismic weight (13,075 kN) of tank.

6.3.5. Moment at Bottom of Wall

Bending moment at the bottom of wall in impulsive mode,

$$M_i = (A_h)_i \, [\, m_i h_i + m_w h_w + m_t h_t \,] \, g$$

(Section 4.7.1.1)

= 0.34 x [(5,42,000 x 1.88)

+ (3,32,824 x 2.65) + 0] x 9.81

= 6,340 kN-m

Similarly, bending moment in convective mode,

$$M_c = (A_h)_c \, m_c \, h_c \, g$$

(Section 4.7.1.1)

$$M_c = (A_h)_c \, m_c \, h_c \, g$$

= 0.06 x 4,85,000 x 2.92 x 9.81

= 875 kN-m

Total bending moment at the bottom of wall,

$$M = \sqrt{M_i^{\,2} + M_c^{\,2}} \qquad \text{(Section 4.7.3)}$$

$$= \sqrt{(6{,}340)^2 + (875)^2}$$

= 6,400 kN-m.

6.3.6. Overturning Moment

Overturning moment at the bottom of base slab in impulsive mode,

$$M_i^* = (A_h)_i \, [\, m_i (h_i^* + t_b) + m_w (h_w + t_b) + m_t (h_t + t_b) + m_b t_b / 2] \, g$$

(Section 4.7.1.2)

= 0.34 x [(5,42,000 x (4.0 + 0.5)) +

(3,32,824 x (2.65 + 0.5)) + 0

+ (2,86,239 x 0.5 / 2)] x 9.81

= 11,870 kN-m.

Similarly, overturning moment in convective mode,

$$M_c^* = (A_h)_c \, m_c \, (h_c^* + t_b) \, g$$

(Section 4.7.1.2)

= 0.06 x 4,85,000 x (4.3 + 0.5) x 9.81

= 1,439 kN-m.

Total overturning moment at the bottom of base slab,

$$M^* = \sqrt{M_i^{*2} + M_c^{*2}} \qquad \text{(Section 4.7.3)}$$

$$= \sqrt{(11{,}870)^2 + (1{,}439)^2}$$

= 11,957 kN-m.

6.3.7. Hydrodynamic Pressure

6.3.7.1. Impulsive Hydrodynamic Pressure

Impulsive hydrodynamic pressure on wall is

$$p_{iw} = Q_{iw}(y) \, (A_h)_i \, \rho \, g \, h$$

$$Q_{iw}(y) = 0.866[1-(y/h)^2] \text{ x } \tanh(0.866 \, L/h)$$

(Section 4.9.1.(b))

At base of wall, $y = 0$;

$Q_{iw}(\, y = 0) = 0.866[1-(0/5)^2]$x tanh(0.866 x10 /5)

= 0.81

Impulsive pressure at the base of wall,

$p_{iw}(\, y = 0\,)$ = 0.81 x 0.34 x 1,000 x 9.81 x 5

= 13.5 kN/m^2.

Impulsive hydrodynamic pressure on the base slab ($y = 0$)

$$p_{ib} = Q_{ib}(x) \, (A_h)_i \, \rho \, g \, h$$

$$Q_{ib}(x) = \sinh(0.866 \, x/L) / \cosh(0.866 \, L/h)$$

(Section 4.9.1(a))

= sinh (0.866 x 10 /10) /cosh (0.866 x 10/5)

= 0.336

Impulsive pressure on top of base slab ($y = 0$)

p_{ib} = 0.336 x 0.34 x 1,000 x 9.81 x 5

= 5.6 kN/m^2

6.3.7.2. Convective Hydrodynamic Pressure

Convective hydrodynamic pressure on wall is

$$p_{cw} = Q_{cw}(y) \, (A_h)_c \, \rho \, g \, L$$

$$Q_{cw}(y) = 0.4165\frac{\cosh\left(3.162\frac{y}{L}\right)}{\cosh\left(3.162\frac{h}{L}\right)}$$

(Section 4.9.2.(b))

At base of wall, $y = 0$;

$$Q_{cw}(\ y = 0\ ) = 0.4165\frac{\cosh\left(3.162\frac{y}{L}\right)}{\cosh\left(3.162\frac{h}{L}\right)}$$

$$= 0.4165 \times \frac{\cosh\left(3.162 \times \frac{0}{10}\right)}{\cosh\left(3.162 \times \frac{5}{10}\right)}$$

$= 0.16$

Convective pressure at the base of wall,

$p_{cw}\ (\ y = 0\ ) = 0.16 \times 0.06 \times 1{,}000 \times 9.81 \times 10$

$= 1.0\ \mathrm{kN/m^2}$

At $y = h$;

$Q_{cw}(y = h) = 0.4165$

Convective pressure at the $y = h$;

$p_{cw}(\ y = h\ ) = 0.4165 \times 0.06 \times 1{,}000 \times 9.81 \times 10$

$= 2.57\ \mathrm{kN/m^2}$.

Convective hydrodynamic pressure on the base slab ($y = 0$)

$p_{cb} = Q_{cb}(x)\ (A_h)_c\ \rho\ g\ D$

$Q_{cb}(x) = 1.25[x/L - 4/3\ (x/L)^3]\ \mathrm{sech}\ (3.162\ h/L)$

(Section 4.9.2(a))

$= 1.25[L/2L - 4/3\ (L/2L)^3]\ \mathrm{sech}\ (3.162 \times 5/10)$

$= 0.165$

Convective pressure on top of base slab ($y = 0$)

$p_{cb} = 0.165 \times 0.06 \times 1{,}000 \times 9.81 \times 10$

$= 1.02\ \mathrm{kN/m^2}$

6.3.8. Pressure Due to Wall Inertia

Pressure on wall due to its inertia,

$p_{ww} = (A_h)_i\ t\ \rho_m\ g$ (Section 4.9.3)

$= 0.34 \times 0.4 \times 25$

$= 3.4\ \mathrm{kN/m^2}$.

This pressure is uniformly distributed along the wall height.

6.3.9. Pressure Due to Vertical Excitation

Hydrodynamic pressure on tank wall due to vertical ground acceleration,

$p_v = (A_v)\ [\rho\ g\ h\ (1 - y\ /\ h\ )]$

(Section 4.10.1)

$$(A_v) = \frac{2}{3}\left(\frac{Z}{2}\frac{I}{R}\frac{S_a}{g}\right)$$

$Z = 0.36$ (IS 1893(Part 1): Table 2; Zone V)

$I = 1.5$ (Table 1)

$R = 2.0$

Time period of vertical mode of vibration is recommended as 0.3 sec in Section 4.10.1, for 5% damping, $S_a/g = 2.5$,

Hence,

$$(A_v) = \frac{2}{3}\times\left(\frac{0.36}{2}\times\frac{1.5}{2.0}\times 2.5\right)$$

$= 0.225$.

At the base of wall, i.e., $y = 0$,

$p_v = 0.225 \times [\ 1 \times 9.81 \times 5 \times (1 - 0/5)]$

$= 11.04\ \mathrm{kN/m^2}$

6.3.10. Maximum Hydrodynamic Pressure

Maximum hydrodynamic pressure,

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}{}^2 + p_v{}^2}$$

(Section 4.10.2)

At the base of wall,

$$p = \sqrt{(13.5 + 3.4)^2 + 1.0^2 + 11.04^2}$$

$= 20.22\ \mathrm{kN/m^2}$.

This maximum hydrodynamic pressure is about 41 % of hydrostatic pressure (49 $\mathrm{kN/m^2}$). This being more than 33%, design of tank will be influenced by hydrodynamic pressure.

6.3.11. Sloshing Wave Height

Maximum sloshing wave height,

$d_{max} = (A_h)_c\ R\ L\ /\ 2$ (Section 4.11)

$= 0.06 \times 2.0 \times 10\ /\ 2$

$= 0.\ 63$ m

6.3.12. Anchorage Requirement

Here, $\frac{h}{L} = \frac{5}{10} = 0.5$;

$$\frac{1}{(A_h)_i} = \frac{1}{0.34} = 2.94$$

As $\frac{h}{L} < \frac{1}{(A_h)_i}$

No anchorage is required.

(Section 4.12)

Thank you

Dr. youssef Hammida