
***IITK-GSDMA* GUIDELINES**

**for SEISMIC DESIGN OF
LIQUID STORAGE TANKS**

Provisions with Commentary and Explanatory Examples



Indian Institute of Technology Kanpur



Gujarat State Disaster Management Authority

October 2007

NATIONAL INFORMATION CENTER OF EARTHQUAKE ENGINEERING

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for SEISMIC DESIGN OF
LIQUID STORAGE TANKS
Provisions with Commentary

Prepared by:
Indian Institute of Technology Kanpur
Kanpur

With Funding by:
Gujarat State Disaster Management Authority
Gandhinagar

October 2007

NATIONAL INFORMATION CENTER OF EARTHQUAKE ENGINEERING
Indian Institute of Technology Kanpur, Kanpur (India)

The material presented in this document is to help educate engineers/designers on the subject. This document has been prepared in accordance with generally recognized engineering principles and practices. While developing this material, many international codes, standards and guidelines have been referred. This document is intended for the use by individuals who are competent to evaluate the significance and limitations of its content and who will accept responsibility for the application of the material it contains. The authors, publisher and sponsors will not be responsible for any direct, accidental or consequential damages arising from the use of material content in this document.

Preparation of this document was supported by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar, through a project at Indian Institute of Technology Kanpur, using World Bank finances. The views and opinions expressed in this document are those of the authors and not necessarily of the GSDMA, the World Bank, or IIT Kanpur.

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Published by:

Coordinator
National Information Center of Earthquake Engineering
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ISBN 81-904190-4-8

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FOREWORD

The earthquake of 26 January 2001 in Gujarat was unprecedented not only for the state of Gujarat but for the entire country in terms of the damages and the casualties. As the state came out of the shock, literally and otherwise, the public learnt for the first time that the scale of disaster could have been far lower had the constructions in the region complied with the codes of practice for earthquake prone regions. Naturally, as Gujarat began to rebuild the houses, infrastructure and the lives of the affected people, it gave due priority to the issues of code compliance for new constructions.

Seismic activity prone countries across the world rely on “codes of practice” to mandate that all constructions fulfill at least a minimum level of safety requirements against future earthquakes. As the subject of earthquake engineering has evolved over the years, the codes have continued to grow more sophisticated. It was soon realized in Gujarat that for proper understanding and implementation, the codes must be supported with commentaries and explanatory handbooks. This will help the practicing engineers understand the background of the codal provisions and ensure correct interpretation and implementation. Considering that such commentaries and handbooks were missing for the Indian codes, GSDMA decided to take this up as a priority item and awarded a project to the Indian Institute of Technology Kanpur for the same. The project also included work on codes for wind loads (including cyclones), fires and terrorism considering importance of these hazards. Also, wherever necessary, substantial work was undertaken to develop drafts for revision of codes, and for development of entirely new draft codes. The entire project is described elsewhere in detail.

The Gujarat State Disaster Management Authority Gandhinagar and the Indian Institute of Technology Kanpur are happy to present the *IITK-GSDMA Guidelines on Seismic Design of Liquid Storage Tanks* to the professional engineering and architectural community in the country. It is hoped that the document will be useful in developing a better understanding of the design methodologies for earthquake-resistant structures, and in improving our codes of practice.

**GSDMA, Gandhinagar
IIT Kanpur**

PREFACE

Liquid storage tanks are commonly used in industries for storing chemicals, petroleum products, etc. and for storing water in public water distribution systems. Importance of ensuring safety of such tanks against seismic loads cannot be overemphasized.

Indian seismic code IS 1893:1984 had some very limited provisions on seismic design of elevated tanks. Compared to present international practice, those provisions of IS 1893:1984 are highly inadequate. Moreover, the code did not cover ground-supported tanks. In 2002, revised Part 1 of IS 1893 has been brought out by the Bureau of Indian Standards (BIS). The other parts, one of which will contain provisions for liquid storage tanks, are yet to be brought out by the BIS.

In the above scenario, to assist the designers for seismic design of liquid storage tanks, it was decided to develop the present document under the project "Review of Building Codes and Preparation of Commentary and Handbooks" assigned by the Gujarat State Disaster Management Authority, Gandhinagar to the Indian Institute of Technology Kanpur in 2003. The provisions included herein are in line with the general provisions of IS1893 (Part 1): 2002 and hence should pose no difficulty to the designers in implementation. To facilitate understanding of the provisions, clause-by-clause commentary is also provided. Further, six explanatory solved examples are provided based on the provisions of these Guidelines.

This document was developed by a team consisting of Professor Sudhir K Jain (Indian Institute of Technology Kanpur) and Professor O R Jaiswal (Visvesvaraya National Institute of Technology, Nagpur). Dr P K Malhotra (FM Global, USA) and Sri L K Jain, (Structural Consultant, Nagpur) reviewed several versions of this document and provided valuable suggestions to improve the same. The document was also placed on the web site of National Information Centre of Earthquake Engineering (www.nicee.org) for comments by the interested professionals and some useful suggestions were provided by Professor A R Chandrasekaran (Hyderabad), Prof K K Khurana (IIT Roorkee), and Sri Rushikesh Trivedi (VMS Consultants, Ahmedabad). Sri Amit Sondeshkar and Ms Shraddha Kulkarni, Technical Assistants at VNIT Nagpur, assisted in development of the solved examples and various graphs and figures of this document.

It is hoped that the designers of liquid retaining tanks will find the document useful. All suggestions and comments are welcome and should be sent to Professor Sudhir K Jain, Department of Civil Engineering, Indian Institute of Technology Kanpur, Kanpur 208 016, e-mail: skjain@iitk.ac.in

SUDHIR K. JAIN
INDIAN INSTITUTE OF TECHNOLOGY KANPUR
OCTOBER 2007

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2.	<i>Elevated Water Tank Supported on 6 Column Staging</i>	250	<i>Staging consists of 6 RC columns; Staging height is 16.3 m with 3 brace levels; Container is of intze type, Seismic zone IV and hard soil strata.</i>	64
3.	<i>Elevated Water Tank Supported on RC Shaft</i>	250	<i>Staging consists of hollow RC shaft of diameter 6.28 m; Shaft height is 16.4 m above ground level; Container is of intze type, Seismic zone IV and hard soil strata</i>	71
4.	<i>Ground Supported Circular Steel Tank</i>	1,000	<i>Steel tank of diameter 12 m and height 10.5 m is resting on ground; Seismic zone V and hard soil strata.</i>	76
5.	<i>Ground Supported Circular Concrete Tank</i>	1,000	<i>Concrete tank of diameter 14 m and height 7 m is resting on ground; Seismic zone IV and soft soil strata</i>	81
6.	<i>Ground Supported Rectangular Concrete Tank</i>	1,000	<i>Rectangular concrete tank with plan dimension 20 x 10 m and height of 5.3 m is resting on ground; Seismic zone V and hard soil strata</i>	84

***IITK-GSDMA* GUIDELINES**
for **SEISMIC DESIGN**
of **LIQUID STORAGE TANKS**

Provisions with Commentary and Explanatory Examples

PART 1: PROVISIONS AND COMMENTARY

PROVISIONS

COMMENTARY

0. – Introduction

0.1 –

In the fifth revision IS 1893 has been split into following five parts:

Part 1: General provisions and buildings

Part 2: Liquid retaining tanks

Part 3: Bridges and retaining walls

Part 4: Industrial structures including stack like structures

Part 5: Dams and embankments

Among these only Part 1, which deals with General Provisions and Buildings has been published by Bureau of Indian Standards. Thus, for design of structures other than buildings, designer has to refer the provisions of previous version of IS 1893 i.e., IS 1893:1984. For seismic design of liquid storage tanks, IS 1893:1984 has very limited provisions. These provisions are only for elevated tanks and ground supported tanks are not considered. Even for elevated tanks, effect of sloshing mode of vibration is not included in IS 1893:1984. Moreover, compared with present international practice for seismic design of tanks, there are many limitations in the provisions of IS 1893:1984, some of which have been discussed by Jain and Medhekar (1993, 1994). Thus, one finds that at present in India there is no proper Code/Standard for seismic design of liquid storage tanks.

In view of non-availability of a proper IS code/standard on seismic design of tanks, present Guidelines is prepared to help designers for seismic design of liquid storage tanks. This Guidelines is written in a format very similar to that of IS code and in future, BIS may as well consider adopting it as IS 1893 (Part 2). Moreover, to be consistent with the present international practice of code writing, a commentary, explaining the rationale behind a particular clause, is also

PROVISIONS

COMMENTARY

provided, wherever necessary. Part 1 of this document contains Guidelines and Commentary. In order to explain the use of this Guidelines, in Part 2, six explanatory examples solved using this Guidelines have been given. These examples include various types of elevated and ground supported tanks. These examples are aimed at explaining use of various clauses given in Guidelines and they may not necessarily cover all the aspects involved in the design of tanks

0.2 –

This Guidelines contains provisions on liquid retaining tanks. Unless otherwise stated, this guideline shall be read necessarily in conjunction with IS: 1893 (Part 1): 2002.

0.3 –

As compared to provisions of IS 1893:1984, in this Guidelines following important provisions and changes have been incorporated:

- a) Analysis of ground supported tanks is included.
- b) For elevated tanks, the single degree of freedom idealization of tank is done away with; instead a two-degree of freedom idealization is used for analysis.
- c) Bracing beam flexibility is explicitly included in the calculation of lateral stiffness of tank staging.
- d) The effect of convective hydrodynamic pressure is included in the analysis.
- e) The distribution of impulsive and convective hydrodynamic pressure is represented graphically for convenience in analysis; a simplified hydrodynamic pressure distribution is also suggested for stress analysis of the tank wall.
- f) Effect of vertical ground acceleration on hydrodynamic pressure is considered.

PROVISIONS

COMMENTARY

0.4 –

In the formulation of this Guidelines, assistance has been derived from the following publications:

1. ACI 350.3, 2001, “Seismic design of liquid containing concrete structures”, American Concrete Institute, Farmington Hill, MI, USA.
2. Eurocode 8, 1998, “Design provisions for earthquake resistance of structures, Part 1- General rules and Part 4 – Silos, tanks and pipelines”, European Committee for Standardization, Brussels.
3. Housner, G. W., 1963a, “Dynamic analysis of fluids in containers subjected to acceleration”, Nuclear Reactors and Earthquakes, Report No. TID 7024, U. S. Atomic Energy Commission, Washington D.C.
4. Housner, G. W., 1963b, “The dynamic behavior of water tanks”, Bulletin of Seismological Society of America, Vol. 53, No. 2, 381-387.
5. Jain, S. K. and Medhekar, M. S., 1993, “Proposed provisions for aseismic design of liquid storage tanks: Part I – Codal provisions”, Journal of Structural Engineering, Vol. 20, No. 3, 119-128.
6. Jain, S. K. and Medhekar, M. S., 1994, “Proposed provisions for aseismic design of liquid storage tanks: Part II – Commentary and examples”, Journal of Structural Engineering, Vol. 20, No. 4, 167-175.
7. Jaiswal, O. R. Rai, D. C. and Jain, S.K., 2004a, “Codal provisions on design seismic forces for liquid storage tanks: a review”, Report No. IITK-GSDMA-EQ-01-V1.0, Indian Institute of Technology, Kanpur.
8. Jaiswal, O. R., Rai, D. C. and Jain, S.K., 2004b, “Codal provisions on seismic analysis of liquid storage tanks: a review” Report No. IITK-GSDMA-EQ-04-V1.0, Indian Institute of Technology, Kanpur.
9. Priestley, M. J. N., et al., 1986, “Seismic design of storage tanks”, Recommendations of a study group of the New Zealand National Society for

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Earthquake Engineering.

10. Veletsos, A. S., 1984, "Seismic response and design of liquid storage tanks", Guidelines for the seismic design of oil and gas pipeline systems, Technical Council on Lifeline Earthquake Engineering, ASCE, N.Y., 255-370, 443-461.

0.5 –

In the formulation of this Guidelines due weightage has been given to international coordination among the standards and practices prevailing in different countries in addition to relating it to the practices in this country.

C0.5 –

Following are some of the international standards and codes of practices which deal with seismic analysis of liquid storage tanks:

1. ACI 350.3, 2001, "Seismic design of liquid containing concrete structures", American Concrete Institute, Farmington Hill, MI, USA.
2. ACI 371-98, 1998, "Guide for the analysis, design, and construction of concrete-pedestal water Towers", American Concrete Institute, Farmington Hill, MI, USA.
3. API 650, 1998, "Welded storage tanks for oil storage", American Petroleum Institute, Washington D. C., USA.
4. AWWA D-100, 1996, "Welded steel tanks for water storage", American Water Works Association, Colorado, USA.
5. AWWA D-103, 1997, "Factory-coated bolted steel tanks for water storage", American Water Works Association, Colorado, USA.
6. AWWA D-110, 1995, "Wire- and strand-wound circular, prestressed concrete water tanks", American Water Works Association, Colorado, USA.
7. AWWA D-115, 1995, "Circular prestressed concrete water tanks with circumferential tendons", American Water Works Association, Colorado, USA.
8. Eurocode 8, 1998, "Design provisions for earthquake resistance of structures, Part 1- General rules and Part 4 – Silos, tanks and pipelines", European committee for Standardization, Brussels.
9. FEMA 368, 2000, "NEHRP recommended provisions for seismic regulations for new buildings and other structures", Building Seismic Safety Council, National Institute of Building Sciences, USA.

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10. IBC 2000, International Building Code International Code Council, Falls Church, Virginia, USA.
11. NZS 3106, 1986, “Code of practice for concrete structures for the storage of liquids”, Standards Association of New Zealand, Wellington.
12. Priestley, M J N, et al., 1986, “Seismic design of storage tanks”, Recommendations of a study group of the New Zealand National Society for Earthquake Engineering.

0.6 –

In the preparation of this Guidelines considerable help has been given by the Indian Institute of Technology Kanpur, Visvesvaraya National Institute of Technology, Nagpur and several other organizations. In particular, the draft was developed through the project entitled *Review of Building Codes and Preparation of Commentary and Handbooks* awarded to IIT Kanpur by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar through World Bank finances.

0.7 –

For the purpose of deciding whether a particular requirement of this Guidelines is complied with, the final value observed or calculated expressing the result of a test or analysis, shall be round off in the accordance with IS: 2-1960. The number of significant places retained in the rounded value should be the same as that of the specified value in this Guidelines.

0.8 –

The units used with the items covered by the symbols shall be consistent throughout this Guidelines, unless specifically noted otherwise.

PROVISIONS

COMMENTARY

1. – Scope

C1. – Scope

1.1 –

This Guidelines covers ground supported liquid retaining tanks and elevated tanks supported on staging. Guidance is also provided on seismic design of buried tanks.

C1.1 –

This Guidelines describes procedure for analysis of liquid containing ground supported and elevated tanks subjected to seismic base excitation. The procedure considers forces induced due to acceleration of tank structure and hydrodynamic forces due to acceleration of liquid.

PROVISIONS

COMMENTARY

2. – References

C2.– References

The following Indian Standards are necessary adjuncts to this Guidelines:

IS No.	Title
456: 2000	Code of Practice for plain and Reinforced Concrete
1893 (Part 1): 2002	Criteria for Earthquake Resistant Design of Structures, Part 1 : General Provisions and Buildings
3370: 1967	Code of Practice for Concrete Structures for the Storage of Liquids
4326: 1993	Code of practice for Earthquake Resistant Design and Construction of Buildings
11682: 1985	Criteria for Design of RCC Staging for Overhead Water Tanks
13920: 1993	Ductile detailing of reinforced concrete structures subjected to seismic forces – Code of practice

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3. – Symbols

The symbols and notations given below apply to the provisions of this Guidelines:

A_h	Design horizontal seismic coefficient
$(A_h)_c$	Design horizontal seismic coefficient for convective mode
$(A_h)_i$	Design horizontal seismic coefficient for impulsive mode
A_v	Design vertical seismic coefficient
B	Inside width of rectangular tank perpendicular to the direction of seismic force
C_c	Coefficient of time period for convective mode
C_i	Coefficient of time period for impulsive mode
d	Deflection of wall of rectangular tank, on the vertical center line at a height \bar{h} when loaded by a uniformly distributed pressure q , in the direction of seismic force
d_{max}	Maximum sloshing wave height
D	Inner diameter of circular tank
E	Modulus of elasticity of tank wall
EL_x	Response quantity due to earthquake load applied in x-direction
EL_y	Response quantity due to earthquake load applied in y-direction
g	Acceleration due to gravity
h	Maximum depth of liquid
\bar{h}	Height of combined center of gravity of half impulsive mass of

C3. – Symbols

a_i, b_i	Values of equivalent linear impulsive pressure on wall at $y = 0$ and $y = h$
a_c, b_c	Values of equivalent linear convective pressure on wall at $y = 0$ and $y = h$

Refer Figure C-3

Refer Figure C-2

F Dynamic earth pressure at rest

Refer Figure C-2 and Clause 4.3.1.2

PROVISIONS

COMMENTARY

	liquid ($m_i / 2$) and mass of one wall (\bar{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	
K_c	Spring stiffness of convective mode	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_s	Lateral stiffness of elevated tank staging	k_h Dynamic coefficient of earth pressure
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	

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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	
\bar{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	

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COMMENTARY

	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	Time period in seconds
T_c	Time period of convective mode (in seconds)		
T_i	Time period of impulsive mode (in seconds)		
V	Total base shear		
		V'	Design base shear at the bottom of base slab/plate of ground supported tank
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	Refer Figure 8a	
y	Vertical distance of a point on tank wall from the bottom of tank wall	Refer Figure 8a	
Z	Seismic zone factor as per Table 2 of IS 1893 (Part 1): 2002		
		γ_s	Density of soil
		μ_c	Convective bending moment coefficient
ρ	Mass density of liquid	μ_i	Impulsive bending moment coefficient
ρ_w	Mass density of tank wall		In SI Units, mass density will be in kg/m^3 , while weight density will be in Newton N/m^3
ϕ	Circumferential angle as described in Figure 8a		
		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

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COMMENTARY

4. – Provisions for Seismic Design

C4.– 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.

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.

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.

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 convective pressure will decrease.

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COMMENTARY

4.2.1 – Ground Supported Tank

C4.2.1 – 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 .

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.

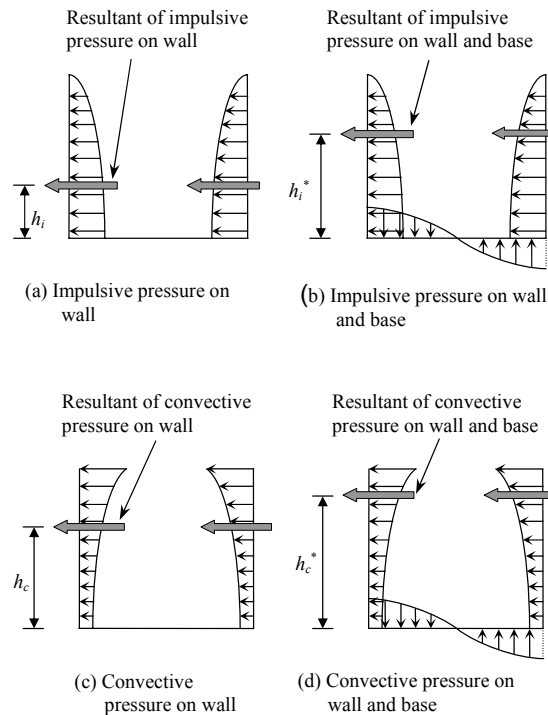


Figure C-1 Qualitative description of hydrodynamic pressure distribution on tank wall and base

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

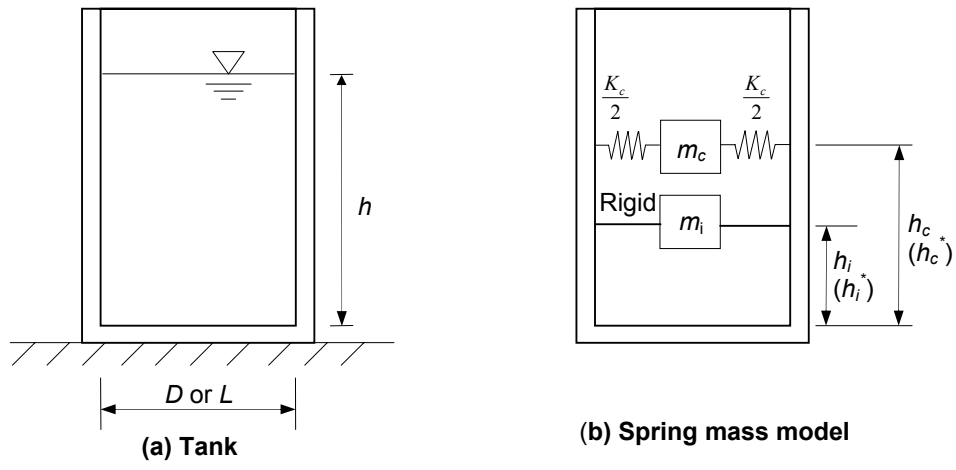


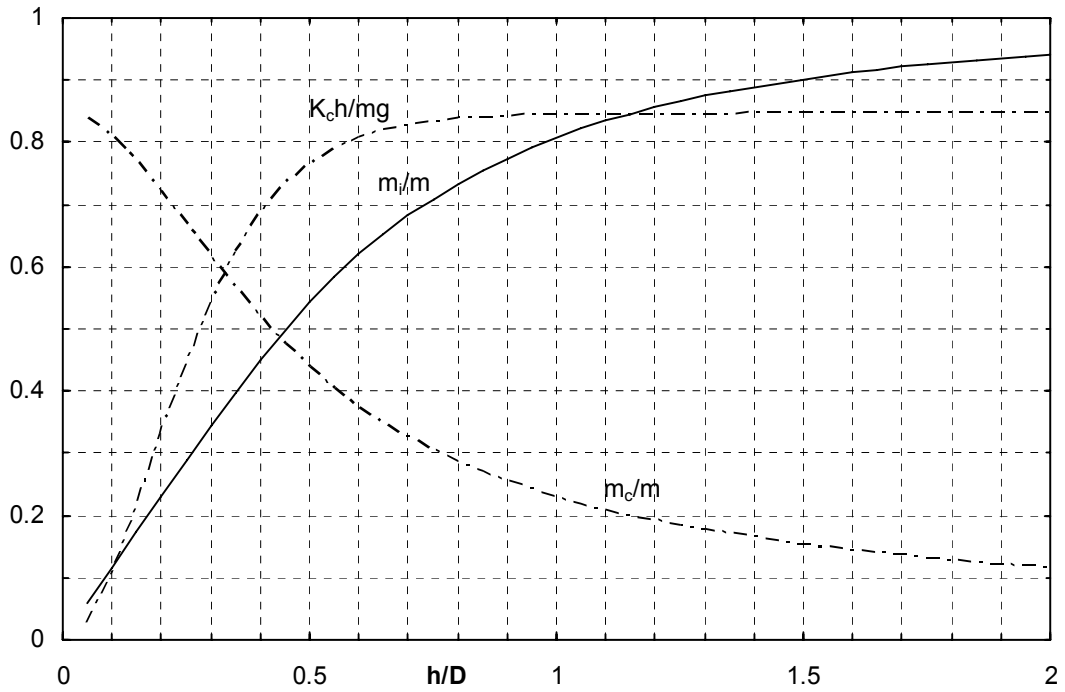
Figure 1 – Spring mass model for ground supported circular and rectangular tank

COMMENTARY

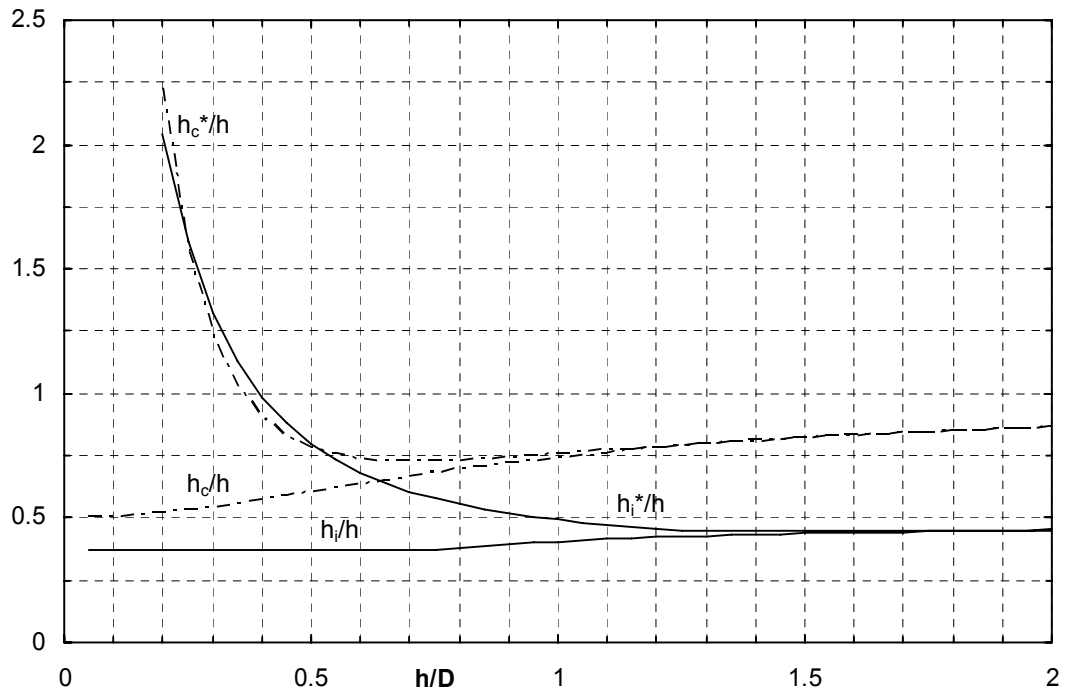
Table C 1 – Expression for parameters of spring mass model

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

PROVISIONS



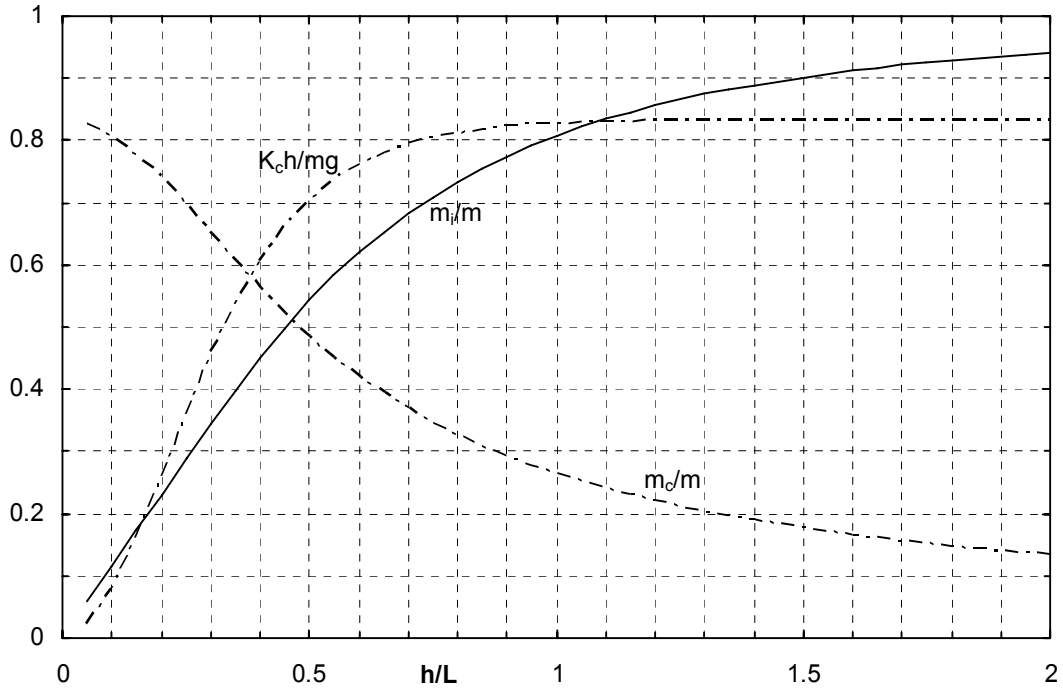
(a) Impulsive and convective mass and convective spring stiffness



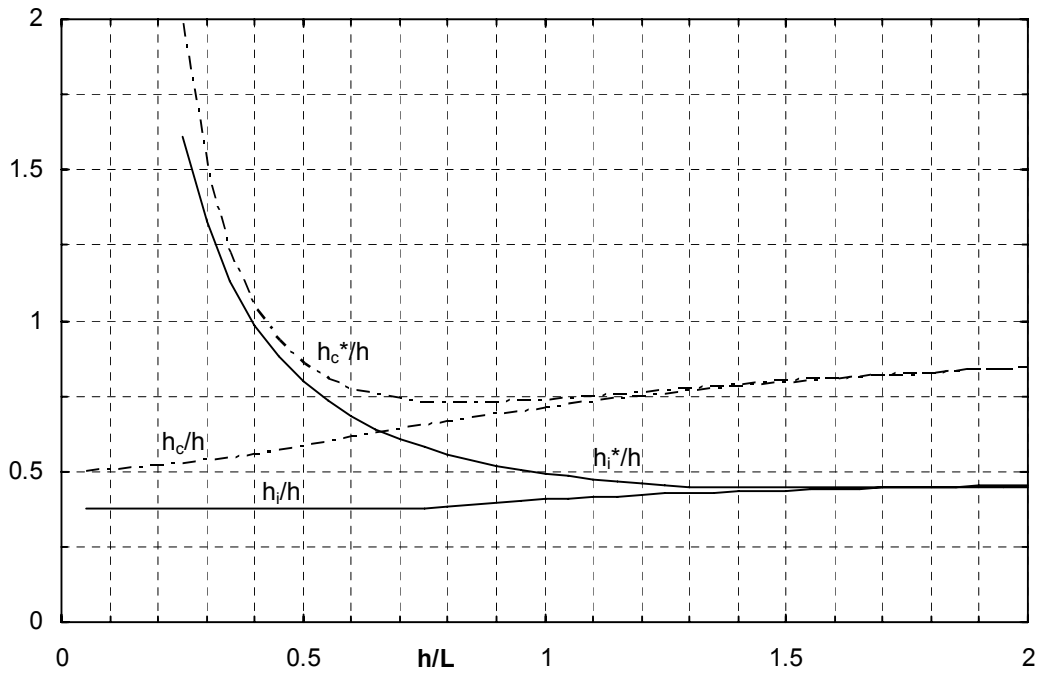
(b) Heights of impulsive and convective masses

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

COMMENTARY

4.2.2 - Elevated Tank

C4.2.2 – Elevated Tank

4.2.2.1 –

Elevated tanks (Figure 4a) can be idealized by a two-mass model as shown in Figure 4c.

C4.2.2.1 –

Most elevated tanks are never completely filled with liquid. Hence a two-mass idealization of the tank is more appropriate as compared to a one-mass idealization, which was used in IS 1893: 1984. Two mass model for elevated tank was proposed by Housner (1963b) and is being commonly used in most of the international codes.

4.2.2.2 –

For elevated tanks with circular container, parameters $m_i, m_c, h_i, h_i^*, h_c, h_c^*$ and K_c shall be obtained from Figure 2. For elevated tanks with rectangular container, these parameters shall be obtained from Figure 3.

C4.2.2.2 –

Please refer commentary of Clause 4.2.1.2 for effect of obstructions inside the container on impulsive and convective mass.

4.2.2.3 –

In Figure 4c, m_s is the structural mass and shall comprise of mass of tank container and one-third mass of staging.

C4.2.2.3 –

Structural mass m_s , includes mass of container and one-third mass of staging. Mass of container comprises of mass of roof slab, container wall, gallery, floor slab, and floor beams.

Staging acts like a lateral spring and one-third mass of staging is considered based on classical result on effect of spring mass on natural frequency of single degree of freedom system (Tse et al., 1983).

4.2.2.4 –

For elevated tanks, the two degree of freedom system of Figure 4c can be treated as two uncoupled single degree of freedom systems (Figure 4d), one representing the impulsive plus structural mass behaving as an inverted pendulum with lateral stiffness equal to that of the staging, K_s and the other representing the convective mass with a spring of stiffness, K_c .

C4.2.2.4 –

The response of the two-degree of freedom system can be obtained by elementary structural dynamics. However, for most elevated tanks it is observed that the two periods are well separated. Hence, the system may be considered as two uncoupled single degree of freedom systems. This method will be satisfactory for design purpose, if the ratio of the period of the two uncoupled systems exceeds 2.5 (Priestley et al. (1986)).

If impulsive and convective time periods are not well separated, then coupled 2-DOF system will have to be solved using elementary structural dynamics. In this context it shall be noted that due to different damping of impulsive and convective components, this 2-DOF system may have non-proportional damping.

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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.

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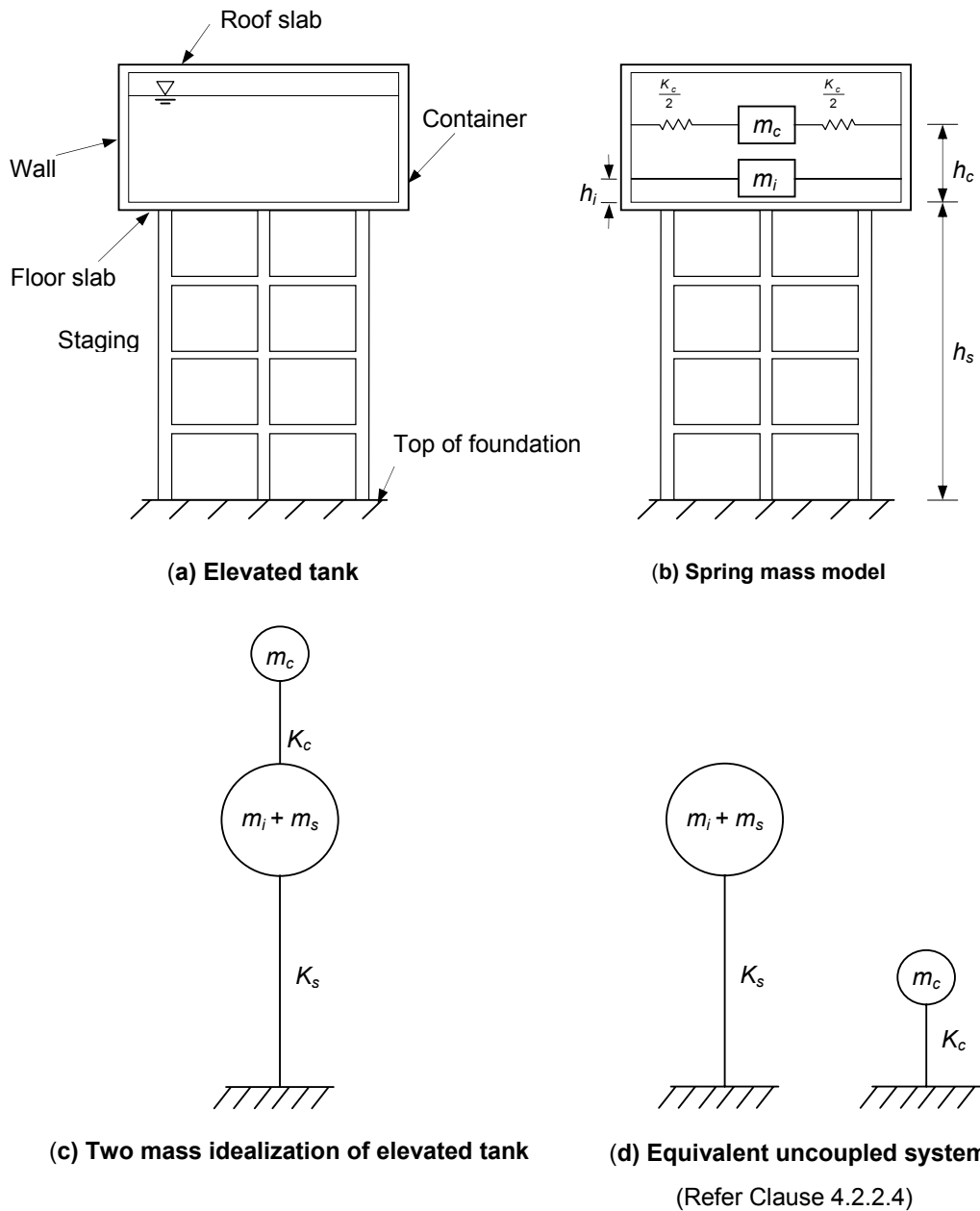


Figure 4 – Two mass idealization for elevated tank

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COMMENTARY

4.3 – Time Period

C4.3 – Time Period

4.3.1 – Impulsive Mode

C4.3.1 – 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

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

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

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 Priestley et al. (1986) also specify the same expression for obtaining time period of rectangular tank.

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

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 ,

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

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

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

B = Inside width of tank.

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

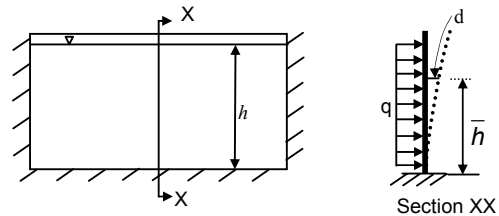
For tanks without roof, deflection, d can be obtained by assuming wall to be free at top and fixed at three edges (Figures C-2a).

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

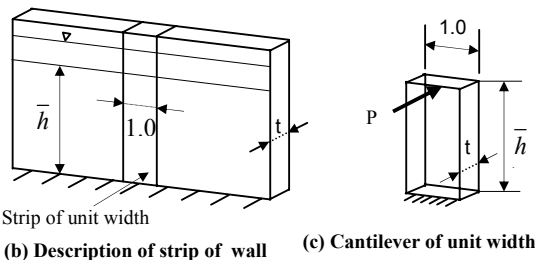
height \bar{h} , which is subjected to concentrated load, $P = q \bar{h}$ (Figures C-2b and C-2c). Thus, for a tank with wall of uniform thickness, one can obtain d as follows:

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

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



Strip of unit width

(b) Description of strip of wall

(c) Cantilever of unit width

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

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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.

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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.

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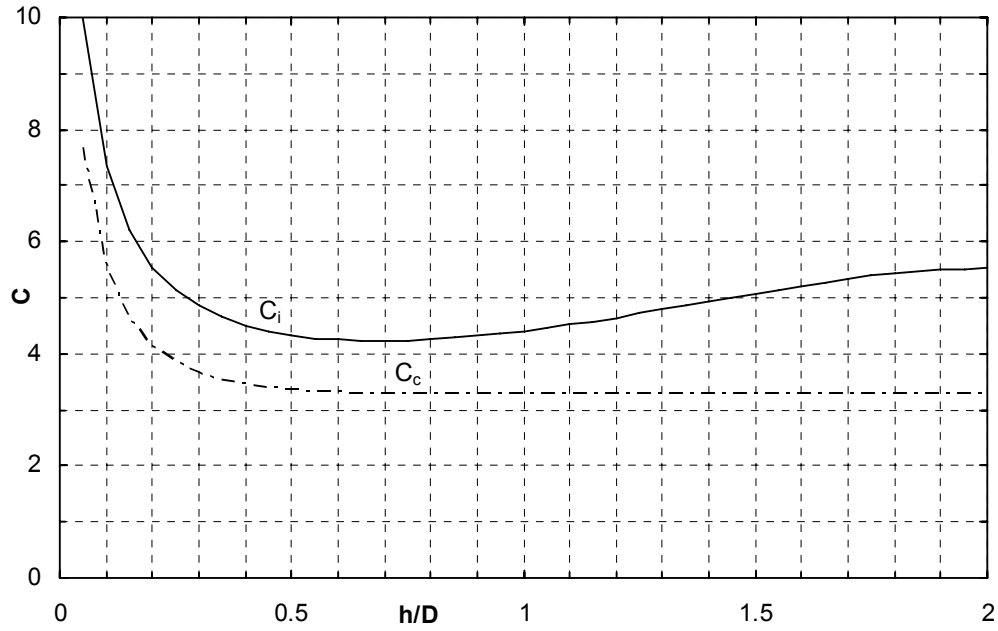


Figure 5 – Coefficient of impulsive (C_i) and convective (C_c) mode time period for circular tank

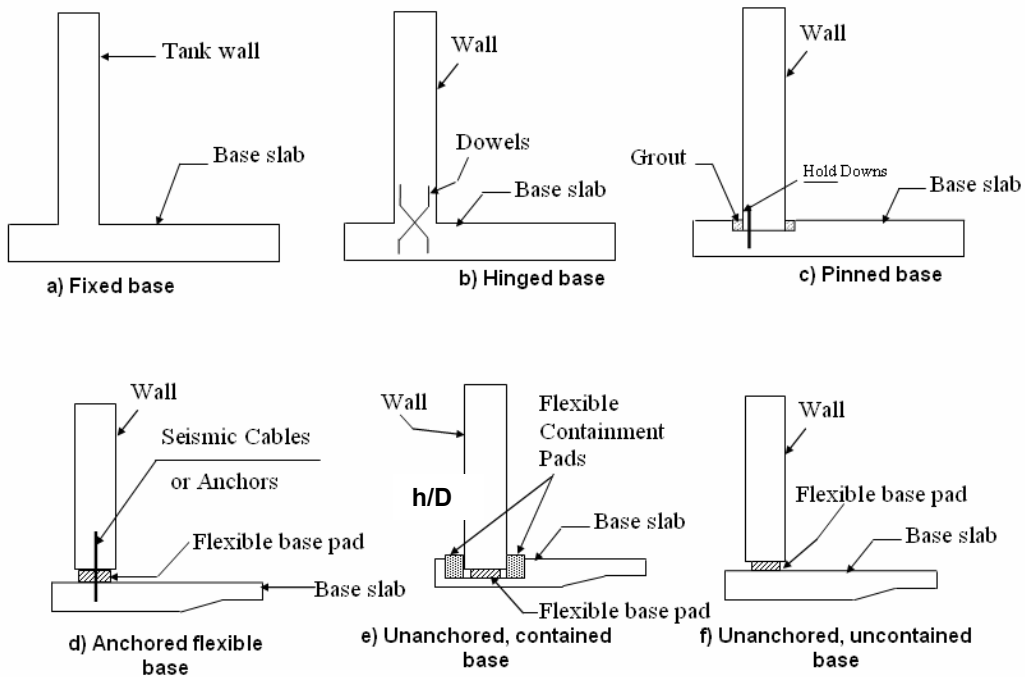


Figure 6 – Types of connections between tank wall and base slab

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COMMENTARY

4.3.2 – Convective Mode

C4.3.2 – 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

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

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direction of seismic force.

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Figure C-3.

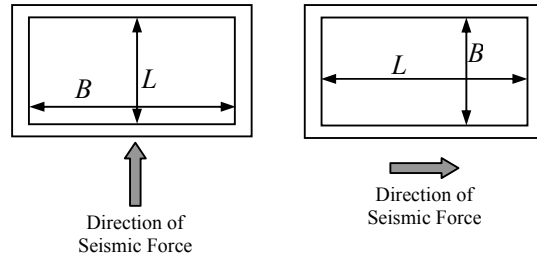


Figure C-3 Description of length, L and breadth, B of rectangular tank

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).

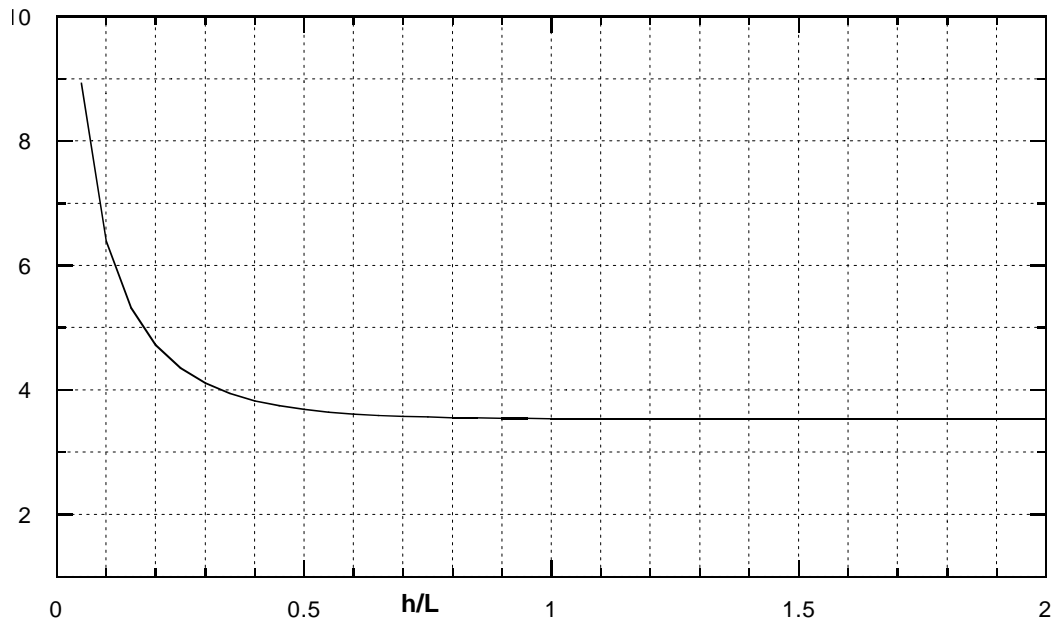


Figure 7 – Coefficient of convective mode time period (C_c) for rectangular tank

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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 possess 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

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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.

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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).

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Table 2 – Response reduction factor, R

Type of tank	R
<u>Elevated tank</u>	
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
<u>Ground supported tank</u>	
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.

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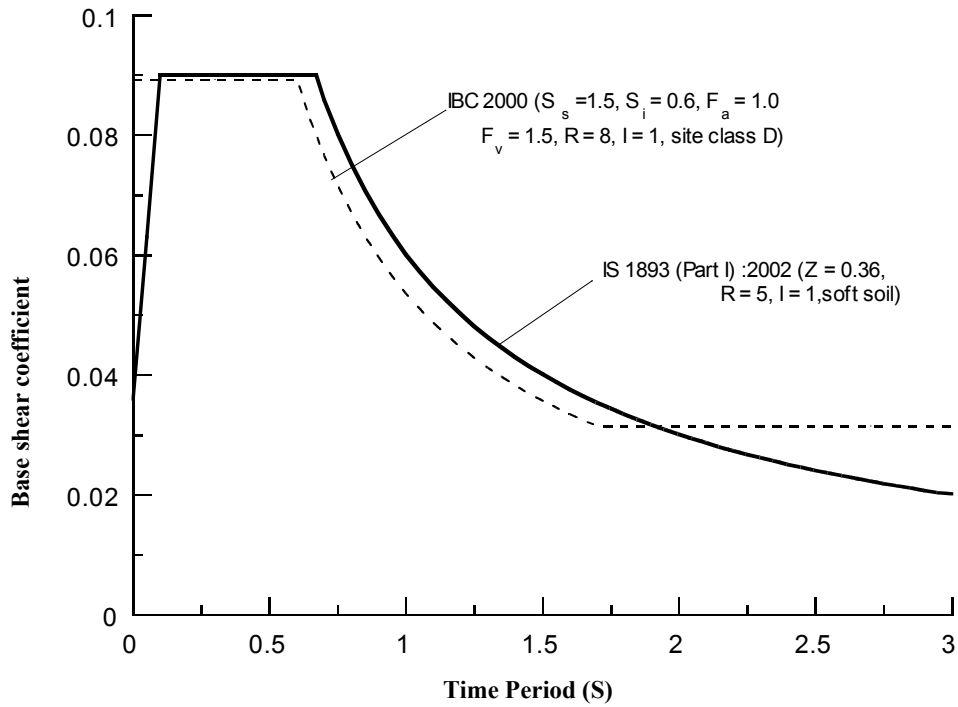


Figure C-4a Comparison of base shear coefficient obtained from IBC 2000 and IS 1893 (Part 1):2002, for a building with special moment resisting frame. IBC values are divided by 1.4 to bring them to working stress level (From Jaiswal et. al., 2004a)

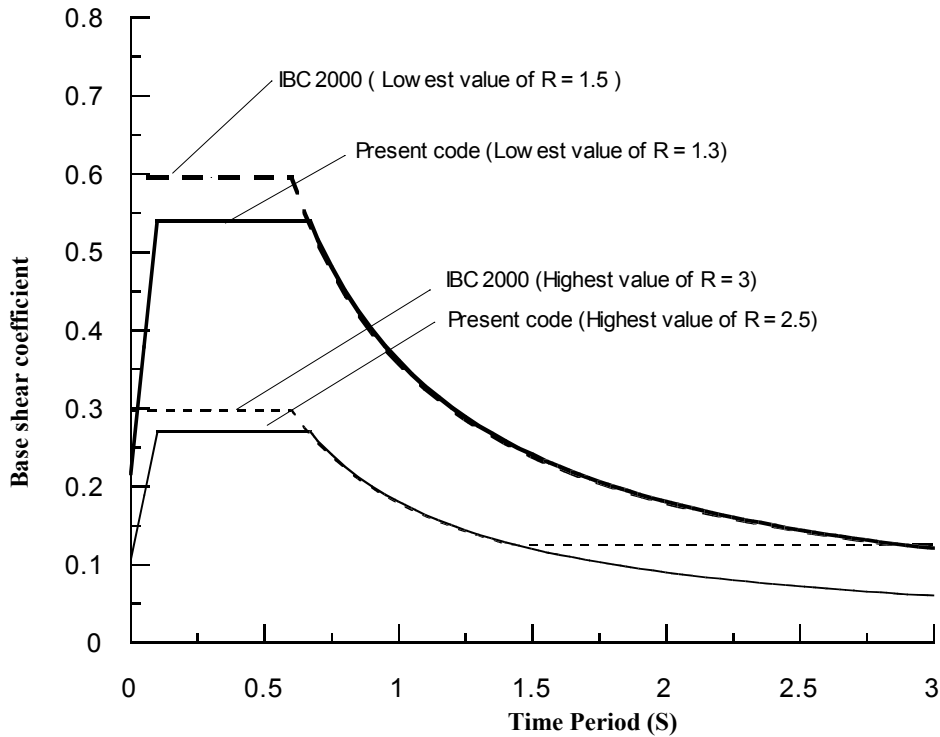


Figure C-4b Comparison of base shear coefficient obtained from IBC 2000 and present code, for tanks with highest and lowest values of R. (From Jaiswal et. al., 2004a)

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

Type of structure	<i>R</i>
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**COMMENTARY****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.

C4.5.3 –

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

For hard soil sites

$$\begin{aligned} S_a/g &= 2.5 && \text{for } T < 0.4 \\ &= 1.0/T && \text{for } T \geq 0.4 \end{aligned}$$

For medium soil sites

$$\begin{aligned} S_a/g &= 2.5 && \text{for } T < 0.55 \\ &= 1.36/T && \text{for } T \geq 0.55 \end{aligned}$$

For soft soil sites

$$\begin{aligned} S_a/g &= 2.5 && \text{for } T < 0.67 \\ &= 1.67/T && \text{for } T \geq 0.67 \end{aligned}$$

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

COMMENTARY

4.6 - Base Shear

C4.6 – 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.

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.

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.

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.

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

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

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

4.7 – Base Moment

C4.7 – Base Moment

4.7.1 – Ground Supported Tank

C4.7.1 – 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.

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

$$M_i = (A_h)_i \left[m_i h_i \mu_i + m_w h_w (1 - y/h)^2 / 2 \right] g$$

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

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.

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

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

and overturning moment in convective mode is given by

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

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

where

m_b = mass of base slab/plate, and

t_b = thickness of base slab/plate.

4.7.2 – Elevated Tank

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

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

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.

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

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

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

4.7.4 –

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

COMMENTARY

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.

C4.7.3 –

See commentary of Clause 4.6.3

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.

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COMMENTARY

4.8 – Direction of Seismic Force

C4.8 – 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.

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.

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.

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.

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:

$$\pm EL_x \pm 0.3 EL_y \text{ and } \pm 0.3 EL_x \pm EL_y$$

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.

C4.8.3 –

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

$$\begin{array}{ll} (EL_x + 0.3 EL_y); & (EL_x - 0.3 EL_y) \\ -(EL_x + 0.3 EL_y); & -(EL_x - 0.3 EL_y) \\ (0.3EL_x + EL_y); & (0.3EL_x - EL_y) \\ -(0.3EL_x + EL_y); & -(0.3EL_x - EL_y) \end{array}$$

COMMENTARY

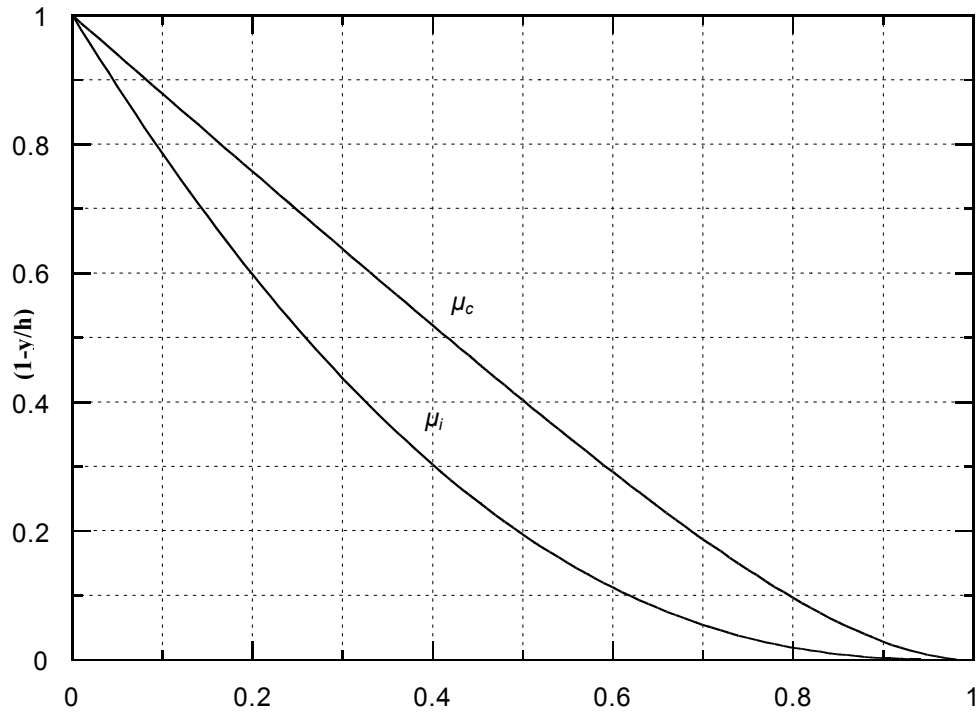


Fig. C- 5 Variation of impulsive and convective bending moment coefficients with height
(From Malhotra, 2004)

PROVISIONS

COMMENTARY

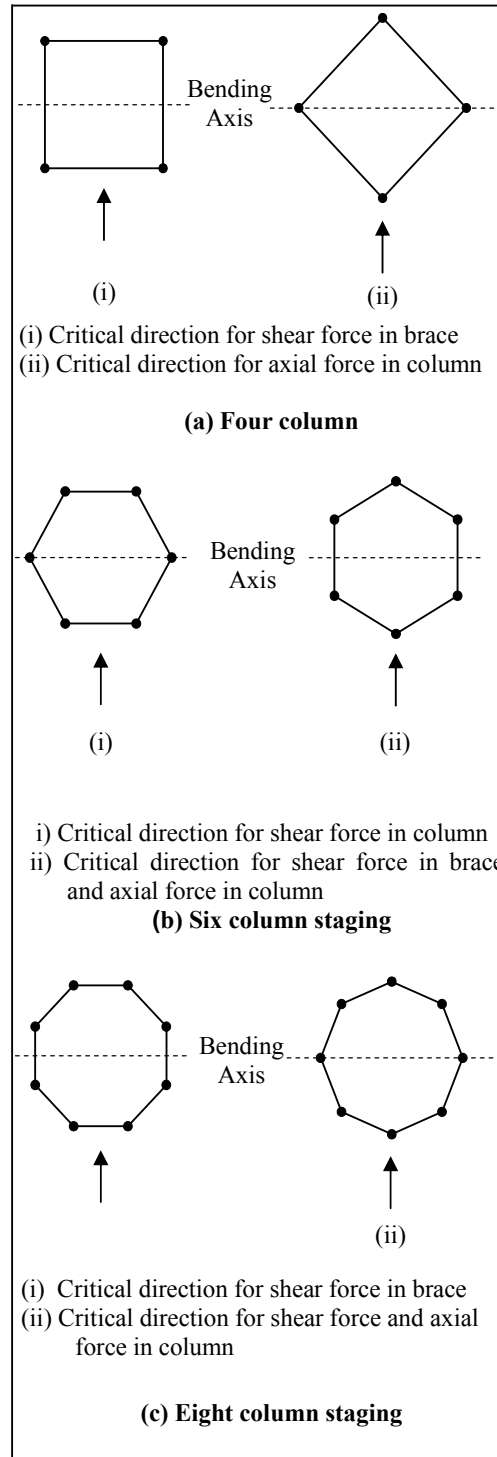


Figure C-6 Critical direction of seismic force for typical frame type staging profiles

PROVISIONS

COMMENTARY

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.

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.

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COMMENTARY

(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

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.

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

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

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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.

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pressure.

For circular tanks, maximum hydrodynamic force per unit circumferential length at $\phi = 0$, 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:

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

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

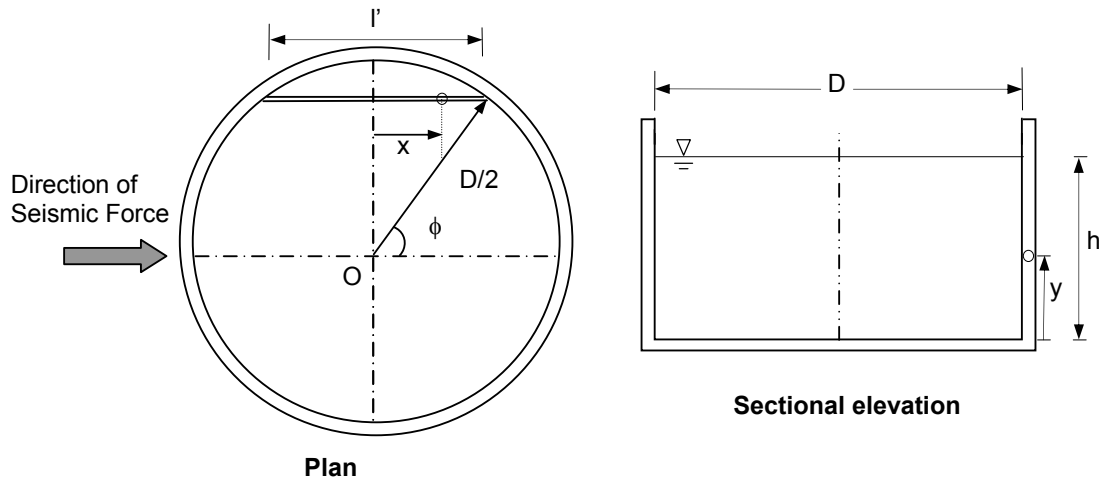
C4.9.5 – Pressure Due to Wall Inertia

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

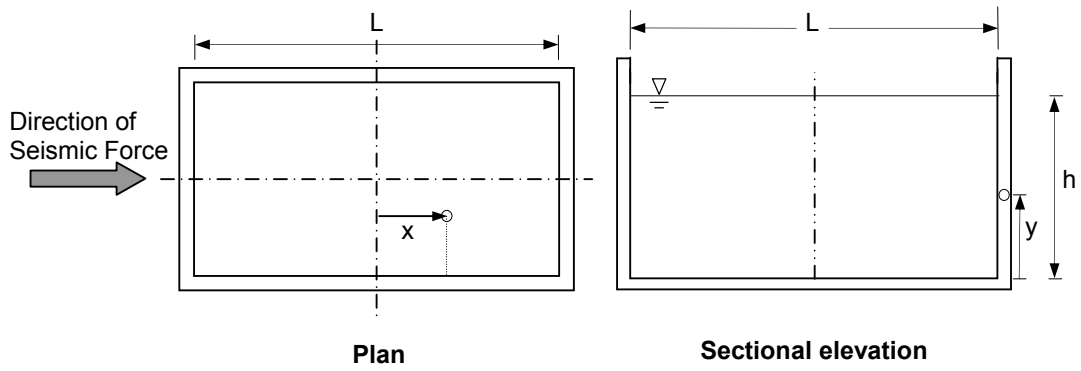
For steel tanks, wall inertia may not be significant. However, for concrete tanks, wall inertia may be substantial.

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

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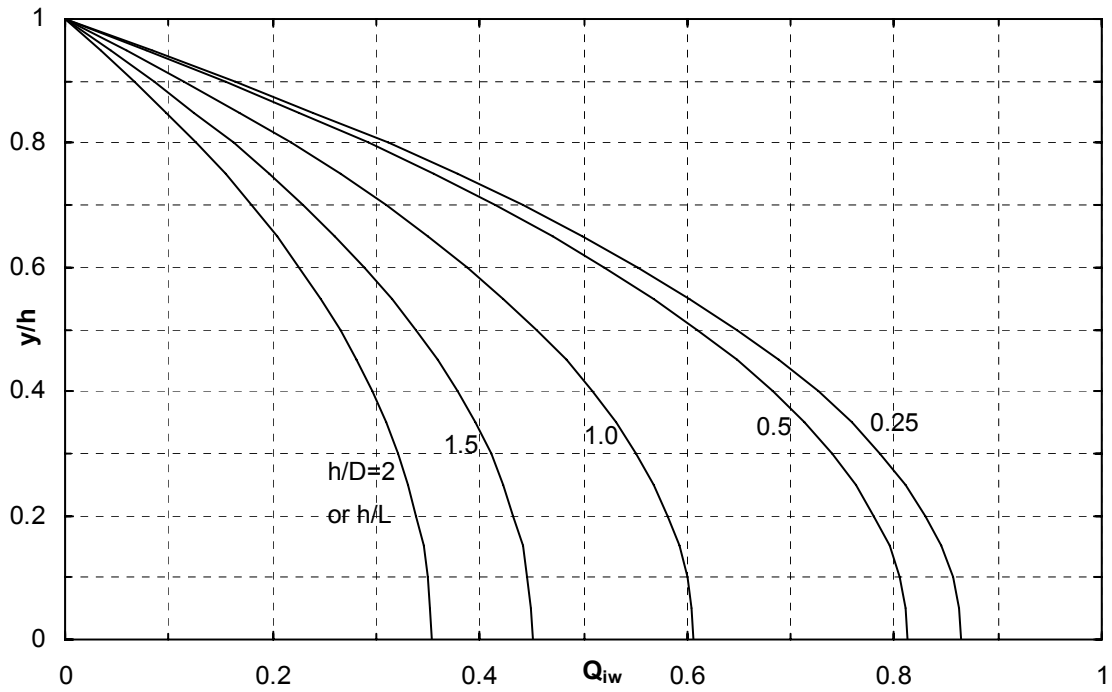
(a) Circular tank



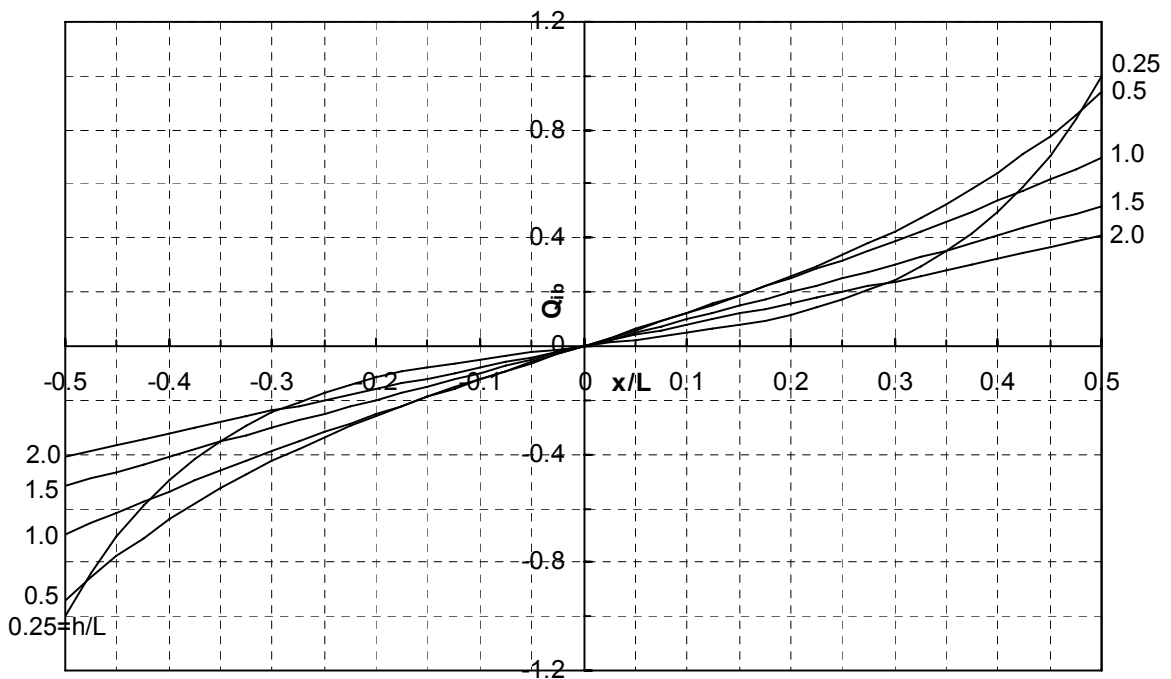
(b) Rectangular tank

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

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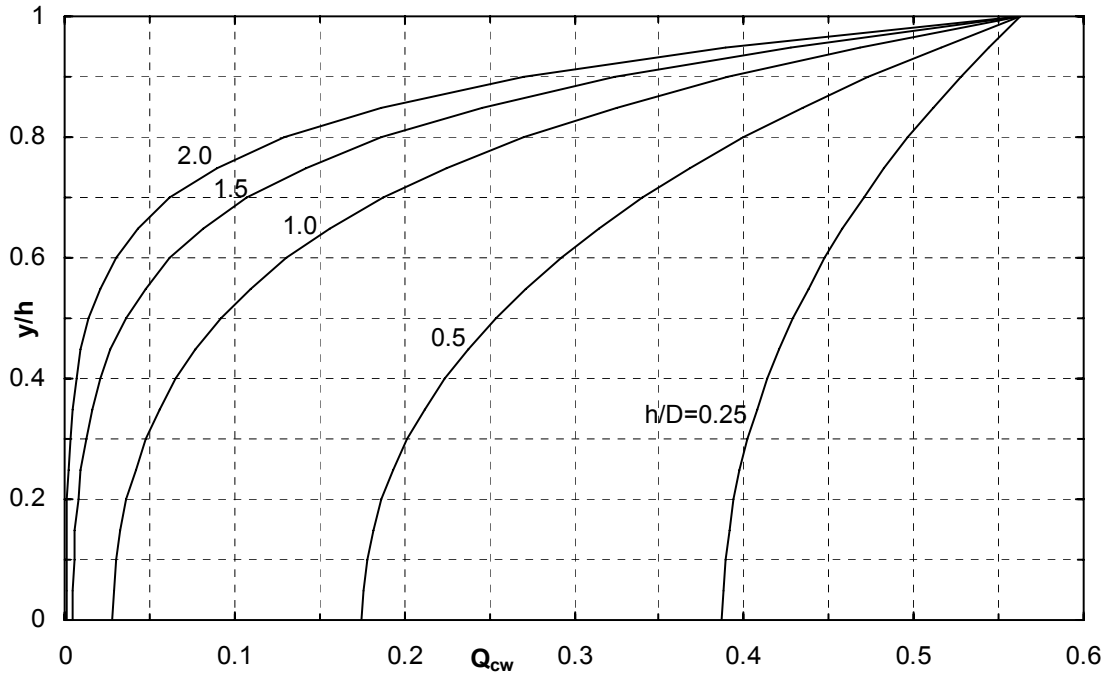
(a) on wall of circular and rectangular tank



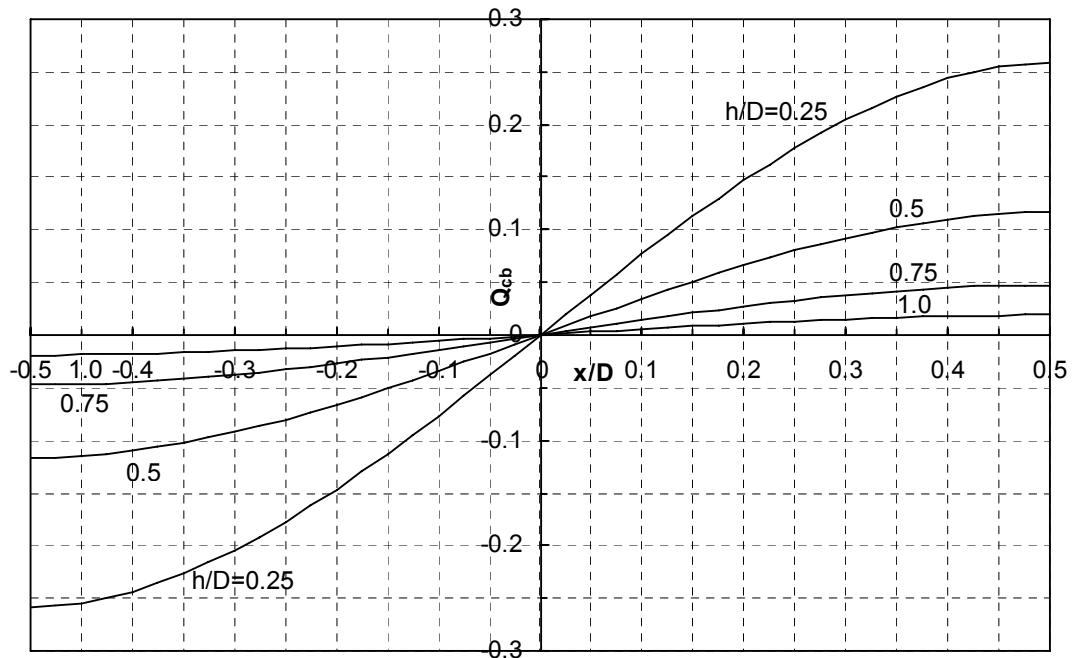
(b) on base of rectangular tank

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

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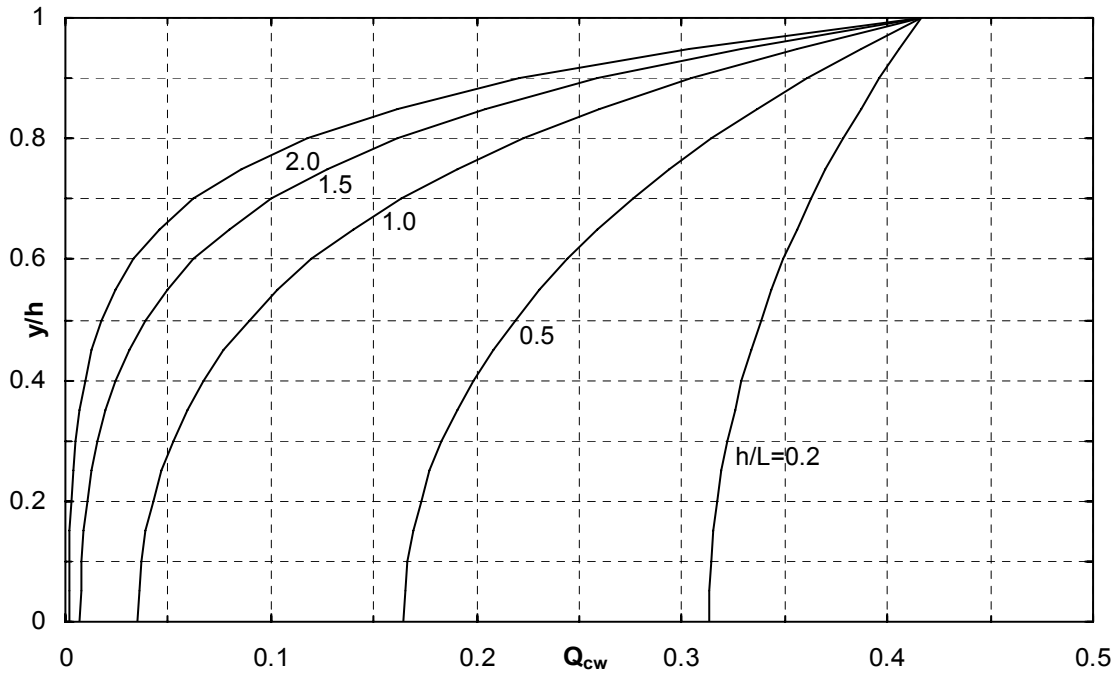
(a) on wall



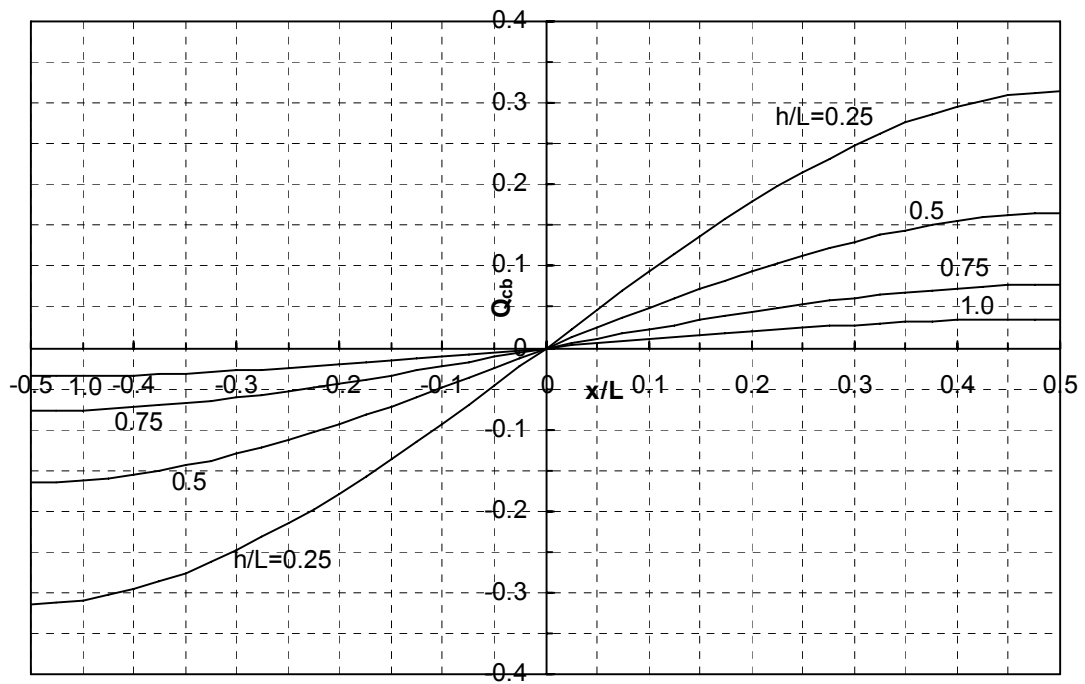
(b) on base

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

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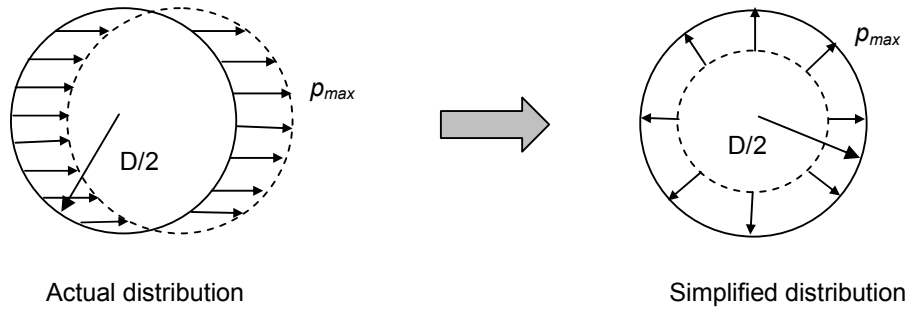
(a) on wall



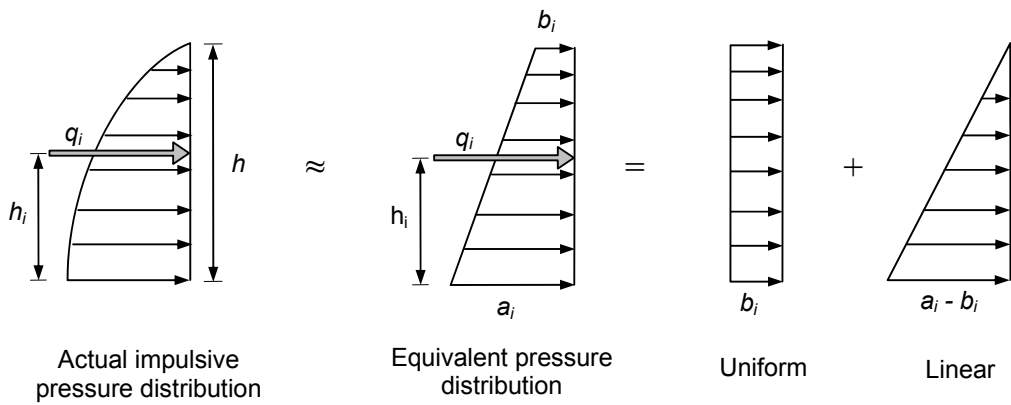
(b) on base

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

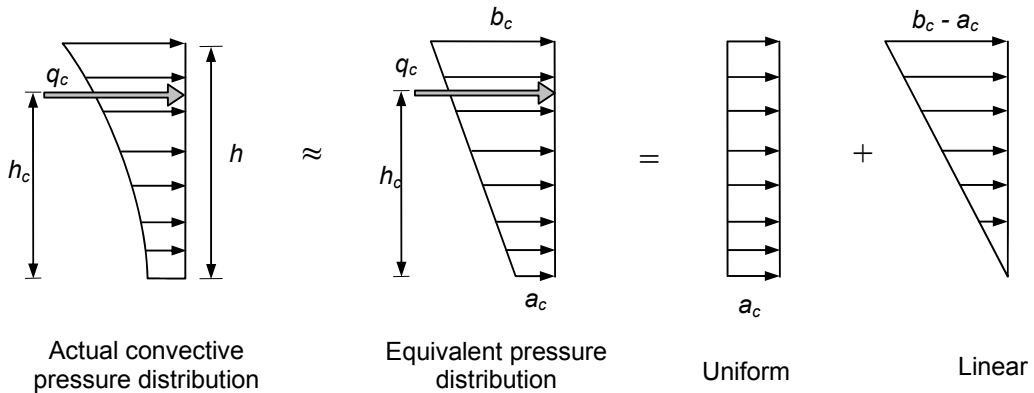
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(a) Simplified pressure distribution in circumferential direction on tank wall



(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

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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

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

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

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 (Part 1).

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.

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4.11 – Sloshing Wave Height

Maximum sloshing wave height is given by

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

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

where

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

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).

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 .

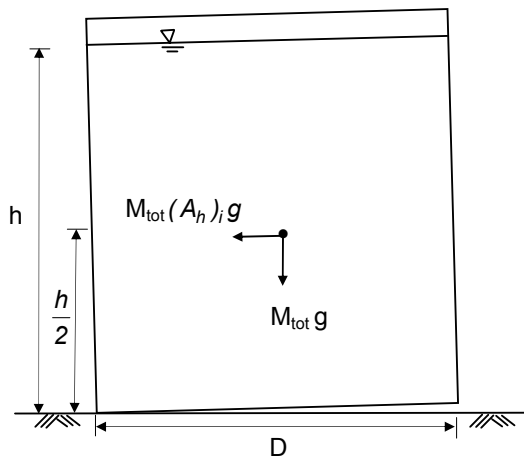


Figure 13 – Initiation of rocking of tank

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} denote 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.

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COMMENTARY

4.13 – Miscellaneous

C4.13 – 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.

C4.13.1 – Piping

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

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.

C4.13.2 – Buckling of Shell

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

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.

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.

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4.13.6 – Quality Control

Quality control in design and constructions are particularly important for elevated tanks in view of several collapses of water tanks during testing. It is necessary that quality of materials and construction tolerances are strictly adhered to during construction phase

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***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

1. Problem Statement:

A RC circular water container of 50 m³ 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/m³. 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

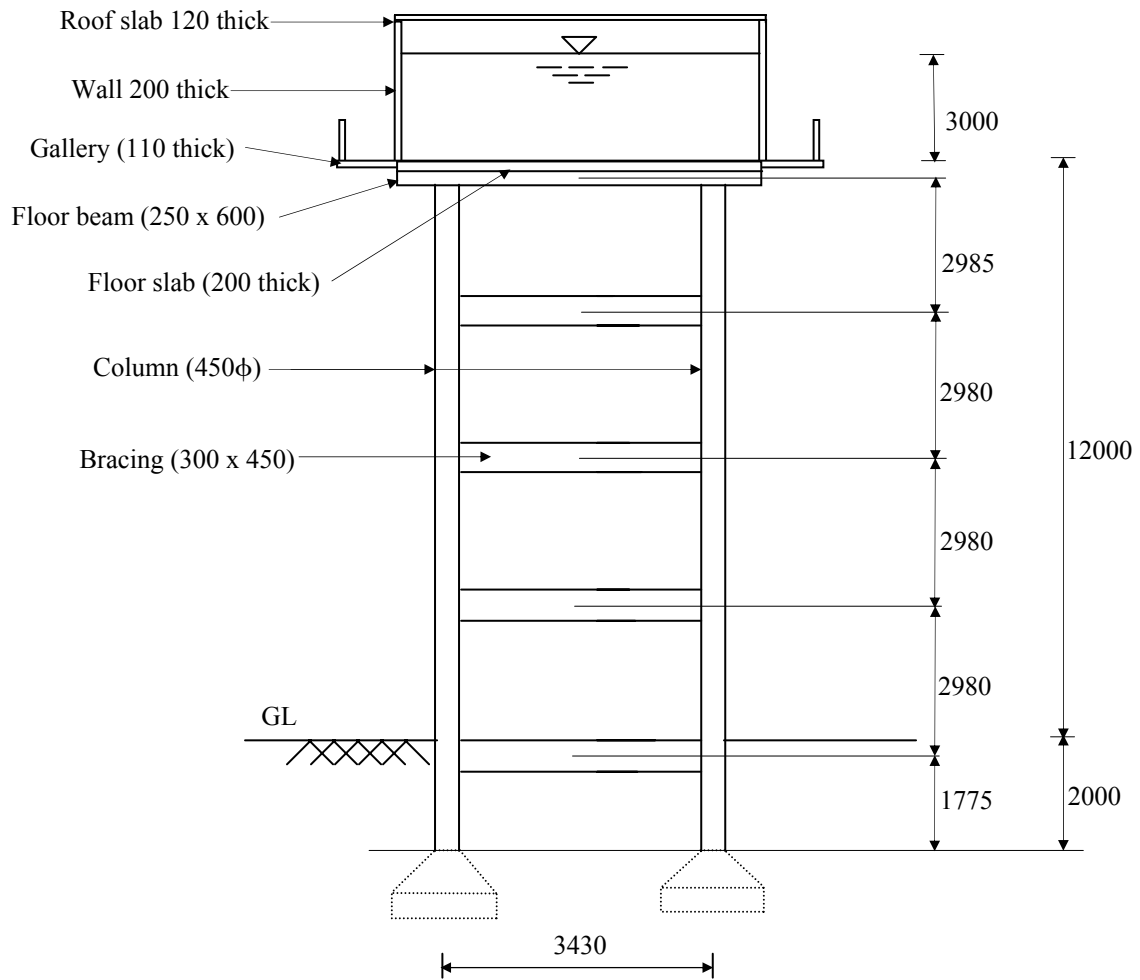
Component	Size (mm)
Roof Slab	120 thick
Wall	200 thick
Floor Slab	200 thick
Gallery	110 thick
Floor Beams	250 x 600
Braces	300 x 450
Columns	450 dia.

1.2. Weight Calculations

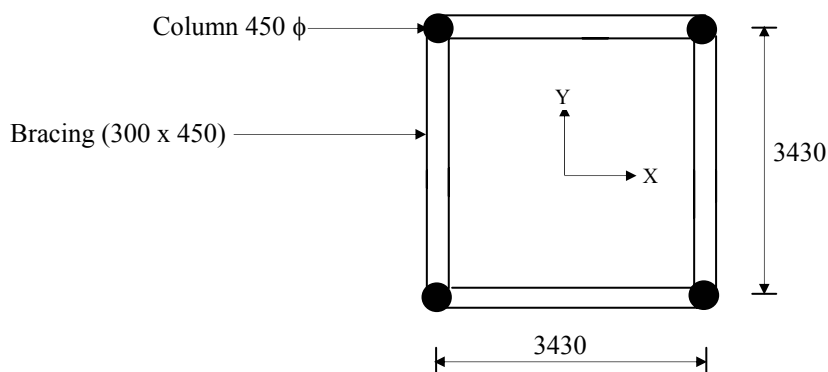
Table 1.2 Weight of various components

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

$$\begin{aligned}
 &= [(60.1 \times 3.36) + (251.4 \times 1.65) \\
 &\quad - (100.2 \times 0.1) - (52.3 \times 0.055) \\
 &\quad - (38.1 \times 0.4)] / 502.1 \\
 &= 1.18 \text{ m.}
 \end{aligned}$$

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

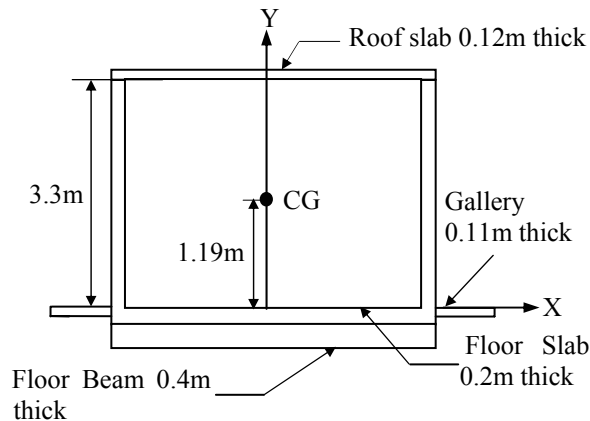


Figure 1.2 CG of empty container

1.4. Parameters of Spring Mass Model

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

Hence, mass of water, $m = 4,99,800 / 9.81$
 $= 50,948 \text{ kg.}$

Depth of water, $h = 3.0 \text{ m.}$

Inner diameter of the tank, $D = 4.65 \text{ m.}$

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

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

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

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

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

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

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

(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

$$\begin{aligned}
 m_s &= (502.1 + 371.3 / 3) \times (1,000 / 9.81) \\
 &= 63,799 \text{ kg.}
 \end{aligned}$$

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 $5,000 \sqrt{f_{ck}} = 5,000 \times \sqrt{20} = 22,360 \text{ MPa}$ or $22.36 \times 10^6 \text{ kN/m}^2$. 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 $= 1.18 + 0.3 = 1.48 \text{ m.}$

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

$$= 10 / 0.00330 = 3,030 \text{ kN/m.}$$

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

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

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.

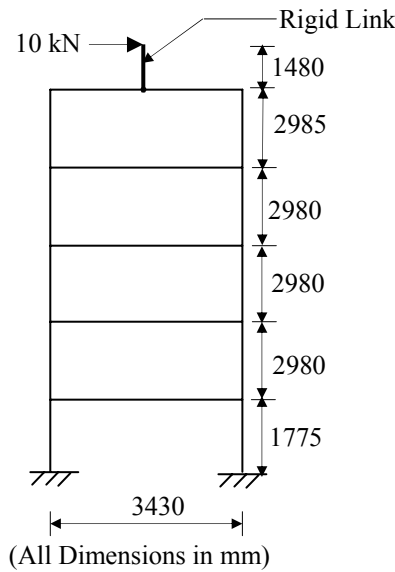


Figure 1.3 FE model of staging

1.6. Time Period

Time period of impulsive mode,

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

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

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))

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

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)

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

$$= 59.9 \text{ kN.}$$

Similarly, base shear in convective mode,

$$V_c = (A_h)_c m_c g \quad (\text{Section 4.6.2})$$

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

$$= 7.0 \text{ kN.}$$

Total base shear at the bottom of staging,

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

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

$$= 60 \text{ kN.}$$

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,

$$M_i^* = (A_h)_i [m_i (h_i^* + h_s) + m_s h_{cg}] g \quad (\text{Section 4.7.2})$$

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

$$= 924 \text{ kN-m.}$$

Similarly, overturning moment in convective mode,

$$M_c^* = (A_h)_c m_c (h_c^* + h_s) g \quad (\text{Section 4.7.2})$$

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

$$= 113 \text{ kN-m.}$$

Total overturning moment at the base of staging,

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

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

$$= 931 \text{ kN-m.}$$

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,

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

$$= 1.41 \text{ kN/m}^2.$$

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

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

(Section 4.9.1(a))

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

$$= 0.95 \text{ kN/m}^2$$

1.10.2. Convective Hydrodynamic Pressure

Convective hydrodynamic pressure on wall,

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

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

(Section 4.9.2(a))

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

At base of wall, $y = 0$;

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

$$= 0.10.$$

Convective pressure at the base of wall,

$$p_{cw}(y = 0) = 0.10 \times 0.04 \times 1,000 \times 9.81 \times 4.65 \times 0.67 \times 1$$

$$= 0.12 \text{ kN/m}^2$$

At $y = h$;

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

Convective pressure at $y = h$,

$$p_{cw}(y = h) = 0.5625 \times 0.04 \times 1,000 \times 9.81 \times 4.65 \times 0.67 \times 1$$

$$= 0.69 \text{ kN/m}^2.$$

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

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

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

(Section 4.9.2(a))

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

$$= 0.07$$

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,

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

$$= 0.06 \times 0.2 \times 25$$

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

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)

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

$$Z = 0.1 \quad (\text{IS 1893(Part 1): Table 2; Zone II})$$

$$I = 1.5 \quad (\text{Table 1})$$

$$R = 2.5$$

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,

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

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

$$= 0.23 \text{ m}.$$

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, $m_s = 63,799 \text{ kg}$.

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

1.15.1. Time Period

Time period of impulsive mode,

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

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

$$= 0.65 \text{ sec}.$$

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,

$$\begin{aligned} V = V_i &= (A_h)_i m_s g && \text{(Section 4.6.2)} \\ &= 0.08 \times 63,799 \times 9.81 \\ &= 50 \text{ kN.} \end{aligned}$$

1.15.4. Base Moment

Total base moment,

$$\begin{aligned} M^* &= (A_h)_i m_s h_{cg} g && \text{(Section 4.7.3)} \\ &= 0.08 \times 63,799 \times 15.18 \times 9.81 \\ &= 760 \text{ kN-m} \end{aligned}$$

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

2. Problem Statement:

An intze shape water container of 250 m³ 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/m³. 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.

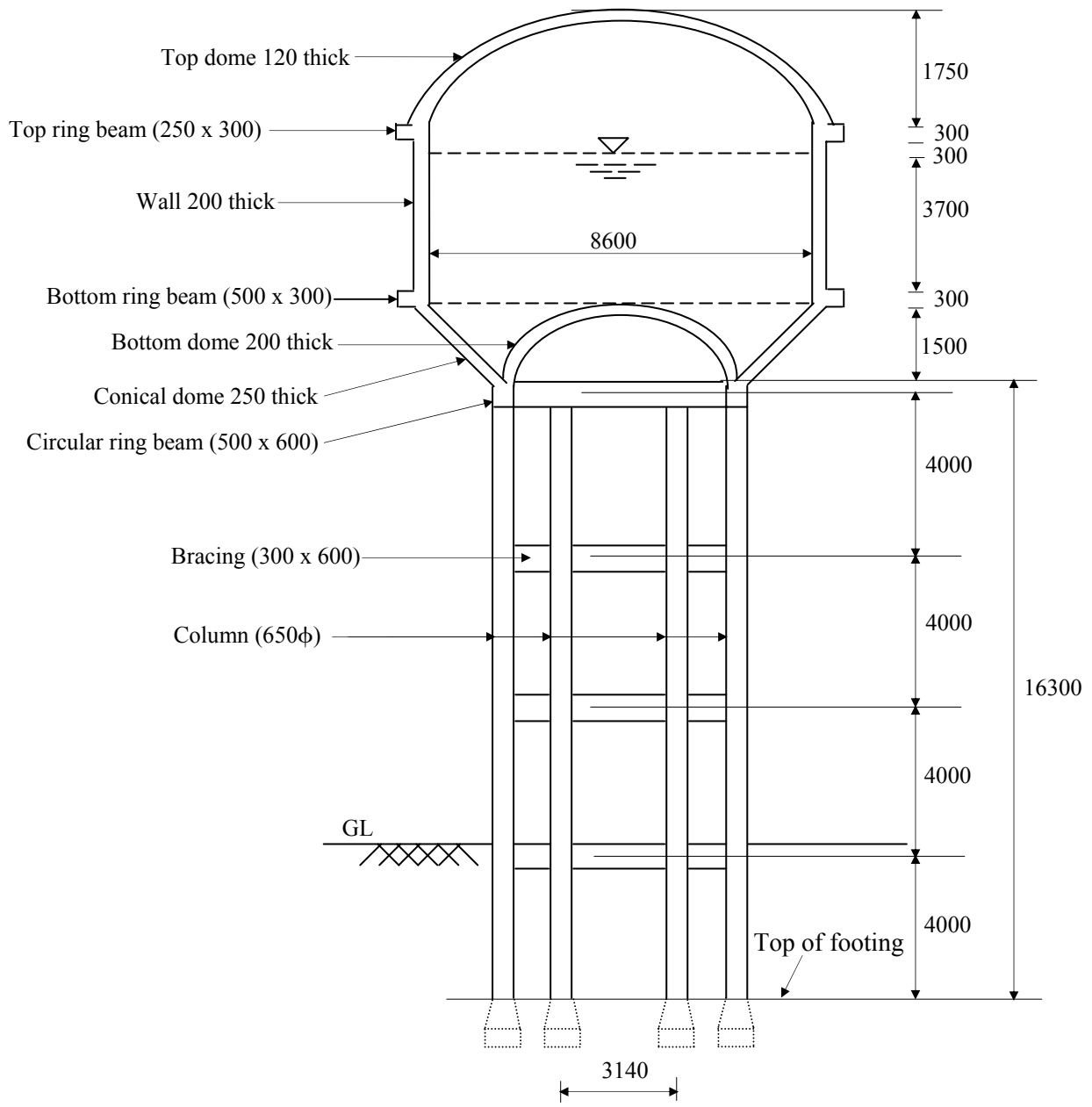
iv) For seismic analysis, free board is not included in depth of water.

From Table 2.2,

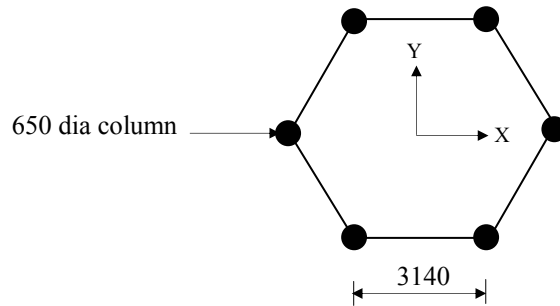
Weight of empty container = $209.3 + 52.1 + 552.9 + 107.2 + 148 + 185.6 + 321.3 = 1,576$ kN

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

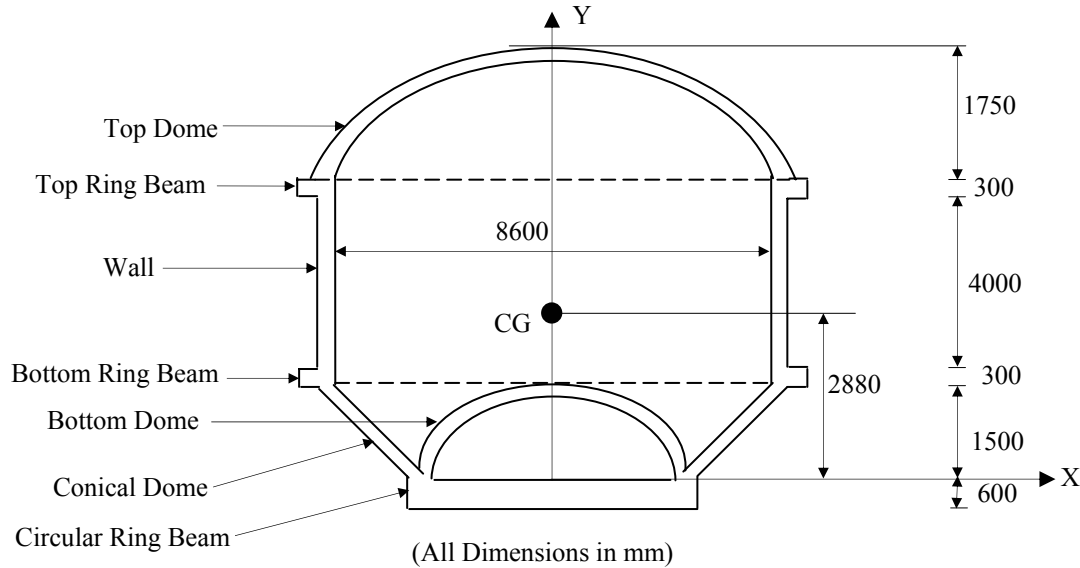
Components of empty container are: top dome, top ring beam, cylindrical wall, bottom ring beam, bottom dome, conical dome and circular ring beam. With reference to Figure 2.2,

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, $h_{cg} = 16.3 + 2.88 = 19.18 \text{ m}$.



(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 m³

Mass of water, $m = 2,55,658 \text{ kg}$.

Inner diameter of tank, $D = 8.6 \text{ m}$.

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,

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

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

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

$$m_i / m = 0.55;$$

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

$$m_c / m = 0.43;$$

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

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

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

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

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

(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,

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

$$= 1,95,821 \text{ kg.}$$

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 $5,000 \sqrt{f_{ck}} = 5,000 \times \sqrt{20} = 22,360 \text{ MPa}$ or $22.36 \times 10^6 \text{ kN/m}^2$. 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 $= 2.88 + 0.3 = 3.18 \text{ m}$.

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

Thus, lateral stiffness of staging, K_s
 $= 10 / (5.616\text{E-}04) = 17,800 \text{ kN/m}$

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.

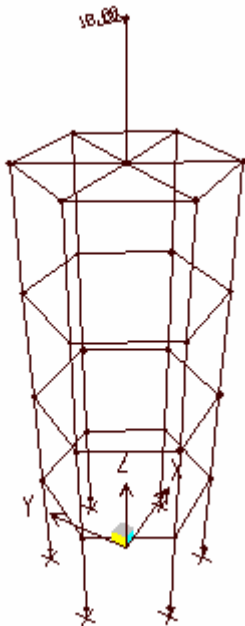


Figure 2.3 FE model of staging

2.6. Time Period

Time period of impulsive mode,

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

$$= 2\pi \sqrt{\frac{1,40,612 + 1,95,821}{178.0 \times 10^5}}$$

$$= 0.86 \text{ sec.}$$

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))

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

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, $T_i = 0.86 \text{ sec}$,

Site has hard soil,

Damping = 5%, (Section 4.4)

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

(IS 1893(Part 1): Figure 2)

$$(A_h)_i = \frac{0.24}{2} \times \frac{1.5}{2.5} \times 1.16 = 0.084$$

Design horizontal seismic coefficient for convective mode,

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

Where,

$$Z = 0.24 \quad (\text{IS 1893(Part 1): Table 2; Zone IV})$$

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

$$\text{Here, } T_c = 3.14 \text{ sec,}$$

Site has hard soil,

$$\text{Damping} = 0.5\%, \quad (\text{Section 4.4})$$

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.

$$(\text{Section 4.5.4})$$

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

2.8. Base Shear

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

$$\begin{aligned} V_i &= (A_h)_i (m_i + m_s) g \quad (\text{Section 4.6.2}) \\ &= 0.084 \times (1,40,612 + 1,95,821) \times 9.81 \\ &= 277 \text{ kN} \end{aligned}$$

Similarly, base shear in convective mode,

$$\begin{aligned} V_c &= (A_h)_c m_c g \quad (\text{Section 4.6.2}) \\ &= 0.040 \times 1,09,933 \times 9.81 \\ &= 43 \text{ kN} \end{aligned}$$

Total base shear at the bottom of staging,

$$\begin{aligned} V &= \sqrt{V_i^2 + V_c^2} \quad (\text{Section 4.6.3}) \\ &= \sqrt{(277)^2 + (43)^2} \\ &= 280 \text{ kN.} \end{aligned}$$

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

Overtopping moment at the base of staging in impulsive mode,

$$\begin{aligned} M_i^* &= (A_h)_i [m_i (h_i^* + h_s) + m_s h_{cg}] g \\ & \quad (\text{Section 4.7.2}) \end{aligned}$$

$$\begin{aligned} &= 0.084 \times [1,40,612 \times (3.43 + 16.3) \\ & \quad + (1,95,821 \times 19.18)] \times 9.81 \\ &= 5,381 \text{ kN-m} \end{aligned}$$

Similarly, overturning moment in convective mode,

$$\begin{aligned} M_c^* &= (A_h)_c m_c (h_c^* + h_s) g \quad (\text{Section 4.7.2}) \\ &= 0.040 \times 1,09,933 \times (3.43 + 16.3) \times 9.81 \\ &= 852 \text{ kN-m} \end{aligned}$$

Total overturning moment,

$$\begin{aligned} M^* &= \sqrt{M_i^{*2} + M_c^{*2}} \quad (\text{Section 4.7.3}) \\ &= \sqrt{(5,381)^2 + (852)^2} \\ &= 5,448 \text{ kN-m.} \end{aligned}$$

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

2.10. Sloshing Wave Height

$$\begin{aligned} d_{max} &= (A_h)_c R D / 2 \quad (\text{Section 4.11}) \\ &= 0.040 \times 2.5 \times 8.6 / 2 \\ &= 0.43 \text{ m.} \end{aligned}$$

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,

$$\begin{aligned} T &= T_i = 2\pi \sqrt{\frac{m_s}{K_s}} \\ &= 2\pi \sqrt{\frac{1,95,821}{178.0 \times 10^3}} = 0.66 \text{ sec} \end{aligned}$$

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 ,

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

Where,

$$Z = 0.24 \quad (\text{IS 1893(Part 1): Table 2; Zone IV})$$

$$I = 1.5 \quad (\text{Table 1})$$

$$R = 2.5 \quad (\text{Table 2})$$

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,

$$V = V_i = (A_h)_i m_s g \quad (\text{Section 4.6.2})$$

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

$$= 211 \text{ kN.}$$

2.11.4. Base Moment

Total base moment,

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

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

$$= 4,053 \text{ kN-m}$$

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

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 = $\pi \times 6.28 \times 0.15 \times 16.4 \times 25 = 1,213 \text{ kN}$
3. Weight of empty container + one third weight of staging = $1,576 + 1,213 / 3 = 1,980 \text{ kN}$
4. Since staging height is 17 m from footing level, height of CG of empty container from top of footing,
 $h_{cg} = 17 + 2.88 = 19.88 \text{ m}$

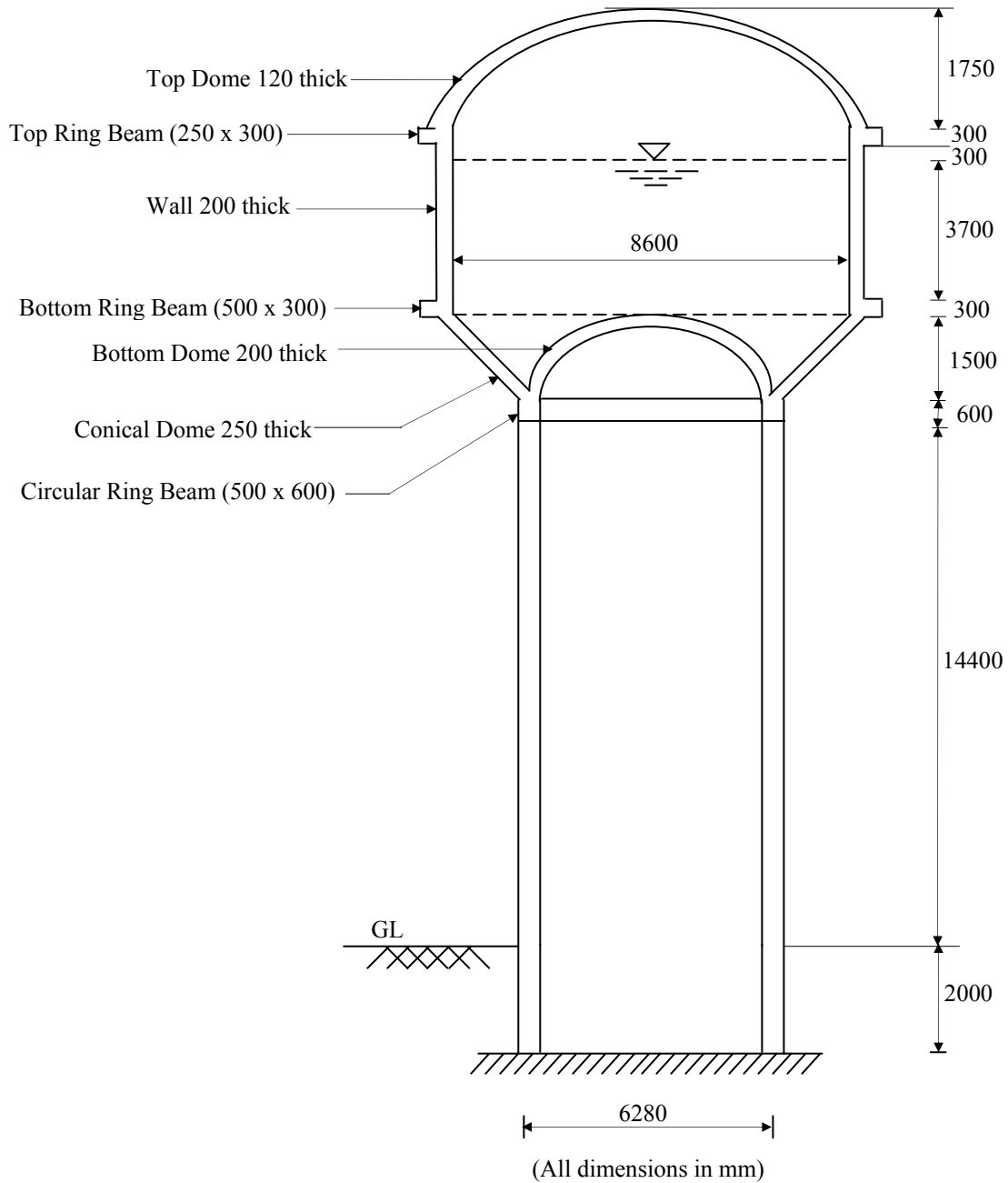


Figure 3.1 Details of tank geometry

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 m³

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,

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

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

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

$$m_i/m = 0.55;$$

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

$$m_c/m = 0.43;$$

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

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

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

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

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

(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,

$$m_s = (1,576 + 1,213/3) \times (1,000/9.81) = 2,01,869 \text{ kg.}$$

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.

$$\text{Lateral Stiffness, } K_s = 3EI/L^3$$

Where,

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

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

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

I = Moment of inertia of shaft cross section

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

$$= 14.59 \text{ m}^4$$

L = Height of shaft

$$= 16.4 \text{ m}$$

Thus,

$$\begin{aligned} \text{Lateral Stiffness} &= 3 \times 22,360 \times 10^6 \times 14.59 / 16.4^3 \\ &= 2.22 \times 10^8 \text{ N/m} \end{aligned}$$

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:

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

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

3.4. Time Period

Time period of impulsive mode,

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

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

$$= 0.25 \text{ sec.}$$

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))

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

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,

$$Z = 0.24 \quad (\text{IS 1893(Part 1): Table 2; Zone IV})$$

$$I = 1.5 \quad (\text{Table 1})$$

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,

$$Z = 0.24 \quad (\text{IS 1893(Part 1): Table 2; Zone IV})$$

$$I = 1.5 \quad (\text{Table 1})$$

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

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

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)

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

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)

$$= 0.25 \times (1,40,612 + 2,01,869) \times 9.81$$

$$= 840 \text{ kN}$$

Similarly, base shear in convective mode,

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

(Section 4.6.2)

$$= 0.06 \times 1,09,933 \times 9.81$$

$$= 65 \text{ kN}$$

Total base shear at the bottom of staging,

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

(Section 4.6.3)

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

$$= 843 \text{ kN.}$$

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)

$$= 0.25 \times [1,40,612 \times (3.43 + 17)$$

$$+ (2,01,869 \times 19.88)] \times 9.81$$

$$= 16,888 \text{ kN-m}$$

Similarly, overturning moment in convective mode,

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

(Section 4.7.2)

$$= 0.06 \times 1,09,933 \times (3.43 + 17) \times 9.81$$

$$= 1,322 \text{ kN-m}$$

Total overturning moment,

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

(Section 4.7.3)

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

$$= 16,940 \text{ kN-m.}$$

3.8. Sloshing Wave Height

Maximum sloshing wave height,

$$\begin{aligned} d_{max} &= (A_h)_c R D / 2 && \text{(Section 4.11)} \\ &= 0.06 \times 1.8 \times 8.6 / 2 \\ &= 0.46 \text{ m} \end{aligned}$$

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,

$$\begin{aligned} T_i &= 2\pi \sqrt{\frac{m_s}{K_s}} \\ &= 2\pi \sqrt{\frac{2,01,869}{2.22 \times 10^8}} \\ &= 0.19 \text{ sec.} \end{aligned}$$

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 ,

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

Where,

$$Z = 0.24$$

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

$$I = 1.5 \quad \text{(Table 1)}$$

$$R = 1.8 \quad \text{(Table 2)}$$

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,

$$\begin{aligned} V &= V_i = (A_h)_i m_s g && \text{(Section 4.6.2)} \\ &= 0.25 \times 2,01,869 \times 9.81 \\ &= 495 \text{ kN} \end{aligned}$$

3.9.4. Base Moment

Total base moment,

$$\begin{aligned} M^* &= (A_h)_i m_s h_{cg} g && \text{(Section 4.7.3)} \\ &= 0.25 \times 2,01,869 \times 19.88 \times 9.81 \\ &= 9,842 \text{ kN-m} \end{aligned}$$

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

4. Problem Statement:

A ground supported cylindrical steel tank with 1,000 m³ 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/m³. Analyze the tank for seismic loads.

Solution:

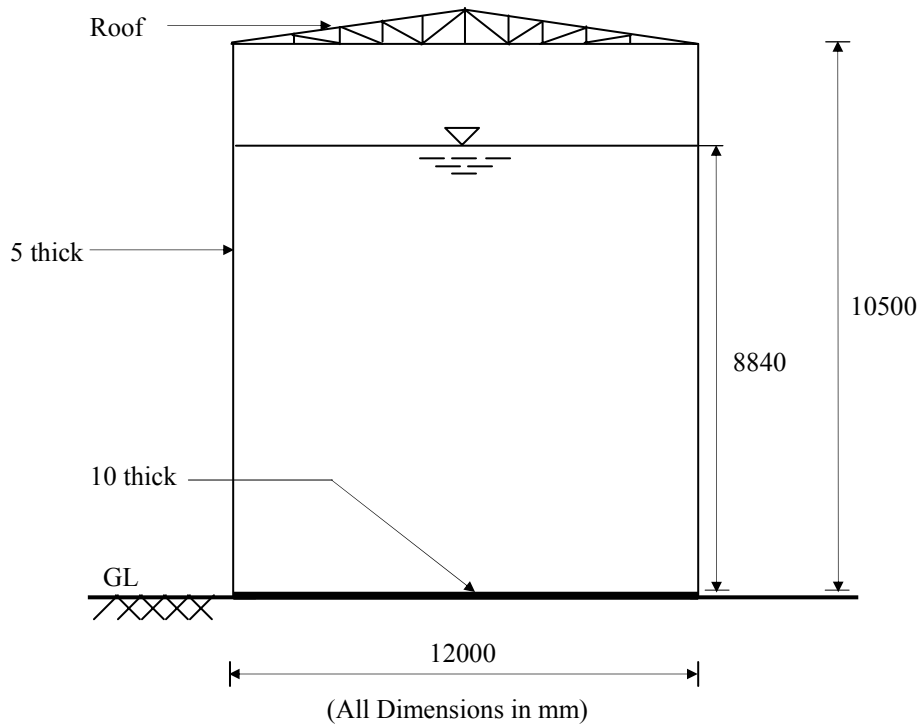


Figure 4.1 Sectional elevation of tank

4.1. Weight Calculations

Weight of tank wall

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

$$= 156 \text{ kN}$$

Mass of tank wall, m_w

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

$$= 15,902 \text{ kg}$$

Weight of base plate

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

$$= 89 \text{ kN}$$

Mass of base plate, m_b

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

$$= 9,072 \text{ kg.}$$

Volume of liquid = 1,000 m³.

Weight of liquid = 9,810 kN

Mass of liquid, $m = 10,00,000 \text{ kg}$

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

Weight of roof = 50 kN

Mass of roof, m_r

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

$$= 5,097 \text{ kg}$$

4.2. Parameters of Spring Mass Model

$h = 8.84 \text{ m}; D = 12 \text{ m}$

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

$m_i / m = 0.703$;

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

$m_c / m = 0.309$

$$m_c = 0.309 \times 10,00,000 = 3,09,000 \text{ kg}$$

$$h_i/h = 0.375 ; \quad h_i = 0.375 \times 8.84 = 3.32 \text{ m}$$

$$h_c/h = 0.677 ; \quad h_c = 0.677 \times 8.84 = 5.98 \text{ m}$$

$$h_i^*/h = 0.587 ; \quad h_i^* = 0.587 \times 8.84 = 5.19 \text{ m}$$

$$h_c^*/h = 0.727 ; \quad h_c^* = 0.727 \times 8.84 = 6.43 \text{ m}$$

(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/m³;

t = Thickness of wall = 0.005 m;

D = Inside diameter of tank = 12 m;

E = Young's modulus for steel = 2×10^{11} N/m²

For $h/D = 0.74$, $C_i = 4.23$

(Section 4.3.1.1)

$$= \frac{4.23 \times 8.84 \times \sqrt{1,000}}{\sqrt{(0.005/12) \times \sqrt{2 \times 10^{11}}}}$$

$$= 0.13 \text{ sec.}$$

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,

$$Z = 0.36 \quad (\text{IS 1893(Part 1): Table 2; Zone V})$$

$$I = 1.5 \quad (\text{Table 1})$$

$$R = 2.5 \quad (\text{Table 2})$$

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

Here, $T_i = 0.13$ sec,

Site has hard soil,

$$\text{Damping} = 5\%, \quad (\text{Section 4.4})$$

$$\text{Hence, } S_a/g = 2.5 \times 1.4 = 3.5$$

(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,

$$Z = 0.36 \quad (\text{IS 1893(Part 1): Table 2; Zone V})$$

$$I = 1.5 \quad (\text{Table 1})$$

$$R = 2.5$$

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,

$$\text{Damping} = 0.5\%, \quad (\text{Section 4.4})$$

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

$$(S_a/g)_c = 1.75 \times 0.275 = 0.48$$

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,

$$V_i = (A_h)_i (m_i + m_w + m_t) g \quad (\text{Section 4.6.1})$$

$$= 0.42 \times (7,03,000 + 15,902 + 5,097) \times 9.81$$

$$= 2,699 \text{ kN}$$

Similarly, base shear in convective mode,

$$V_c = (A_h)_c m_c g \quad (\text{Section 4.6.1})$$

$$= 0.05 \times 3,09,000 \times 9.81$$

$$= 152 \text{ kN}$$

Total base shear at the bottom of wall,

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

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

$$= 2,703 \text{ kN.}$$

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,

$$M_i = (A_h)_i [m_i h_i + m_w h_w + m_t h_t] g \quad (\text{Section 4.7.1.1})$$

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

$$= 9,211 \text{ kN-m}$$

Similarly, bending moment in convective mode,

$$M_c = (A_h)_c m_c h_c g \quad (\text{Section 4.7.1.1})$$

$$= 0.05 \times 3,09,000 \times 5.98 \times 9.81$$

$$= 906 \text{ kN-m}$$

Total bending moment at bottom of wall,

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

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

$$= 9,255 \text{ kN-m.}$$

4.7. Overturning Moment

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

$$M_i^* = (A_h)_i [m_i (h_i^* + t_b) + m_w (h_w + t_b) + m_t (h_t + t_b) + m_b t_b / 2] g \quad (\text{Section 4.7.1.2})$$

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

$$= 14,139 \text{ kN-m.}$$

Similarly, overturning moment in convective mode,

$$M_c^* = (A_h)_c m_c (h_c^* + t_b) g \quad (\text{Section 4.7.1.2})$$

$$= 0.05 \times 3,09,000 \times (6.43 + 0.01) \times 9.81$$

$$= 976 \text{ kN-m.}$$

Total overturning moment at the bottom of base plate,

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

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

$$= 14,173 \text{ kN-m.}$$

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,

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

$$= 23.73 \text{ kN/m}^2.$$

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

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

(Section 4.9.1(a))

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

$$= 15.07 \text{ kN/m}^2$$

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,

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

$$= 0.28 \text{ kN/m}^2$$

At $y = h$;

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

Convective pressure at $y = h$,

$$p_{cw}(y = h) = 0.5625 \times 0.05 \times 1,000 \times 9.81 \times 12 \times 0.67 \times 1$$

$$= 2.22 \text{ kN/m}^2.$$

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 ($y = 0$)

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

$$= 0.30 \text{ kN/m}^2$$

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,

$$q_i = \frac{(A_h)_i m_i g}{\pi D / 2} = \frac{0.38 \times 7,03,000 \times 9.81}{\pi \times 12 / 2}$$

$$= 139.0 \text{ kN/m}$$

Pressure at bottom and top is given by,

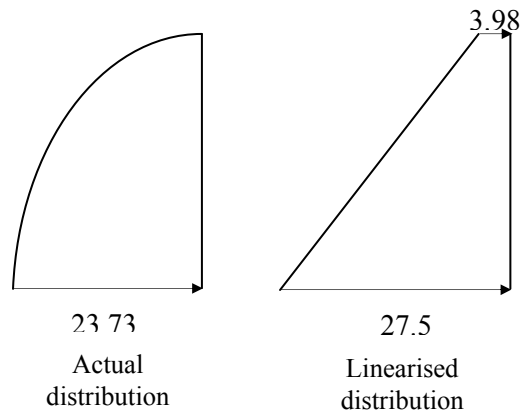
$$a_i = \frac{q_i}{h^2} (4h - 6h_i) = \frac{139.0}{8.84^2} (4 \times 8.84 - 6 \times 3.32)$$

$$= 27.5 \text{ kN/m}^2$$

$$b_i = \frac{q_i}{h^2} (6h_i - 2h) = \frac{139.0}{8.84^2} (6 \times 3.32 - 2 \times 8.84)$$

$$= 3.98 \text{ kN/m}^2$$

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

$$q_c = \frac{(A_h)_c m_c g}{\pi D / 2} = \frac{0.05 \times 3,09,000 \times 9.81}{\pi \times 12 / 2}$$

$$= 8.04 \text{ kN/m}$$

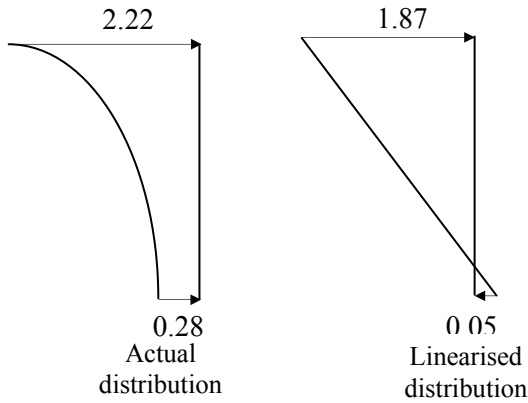
Pressure at bottom and top is given by,

$$a_c = \frac{q_c}{h^2} (4h - 6h_c) = \frac{8.04}{8.84^2} (4 \times 8.84 - 6 \times 5.98)$$

$$= -0.05 \text{ kN/m}^2$$

$$b_c = \frac{q_c}{h^2} (6h_c - 2h) = \frac{8.04}{8.84^2} (6 \times 5.98 - 2 \times 8.84) = 1.87 \text{ kN/m}^2$$

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,

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

$$= 0.38 \times 0.005 \times 78.53$$

$$= 0.15 \text{ kN / m}^2$$

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,

$$p_v = (A_v) [\rho g h (1 - y/h)] \quad (\text{Section 4.10.1})$$

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

$$Z = 0.36 \quad (\text{IS 1893(Part 1): Table 2; Zone V})$$

$$I = 1.5 \quad (\text{Table 1})$$

$$R = 2.5$$

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,

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}^2 + p_v^2} \quad (\text{Section 4.10.2})$$

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,

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

$$= 0.05 \times 2.5 \times 12 / 2$$

$$= 0.75 \text{ m}$$

4.13. Anchorage Requirement

$$\text{Here, } \frac{h}{D} = \frac{8.84}{12} = 0.74 ;$$

$$\frac{1}{(A_h)_i} = \frac{1}{0.38} = 2.63$$

$$\text{As } \frac{h}{D} < \frac{1}{(A_h)_i}$$

No anchorage is required.

(Section 4.12)

Example 5 – Ground Supported Circular Concrete Tank

5. Problem Statement:

A ground supported cylindrical RC water tank without roof has capacity of 1,000 m³. 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/m³. Analyze the tank for seismic loads.

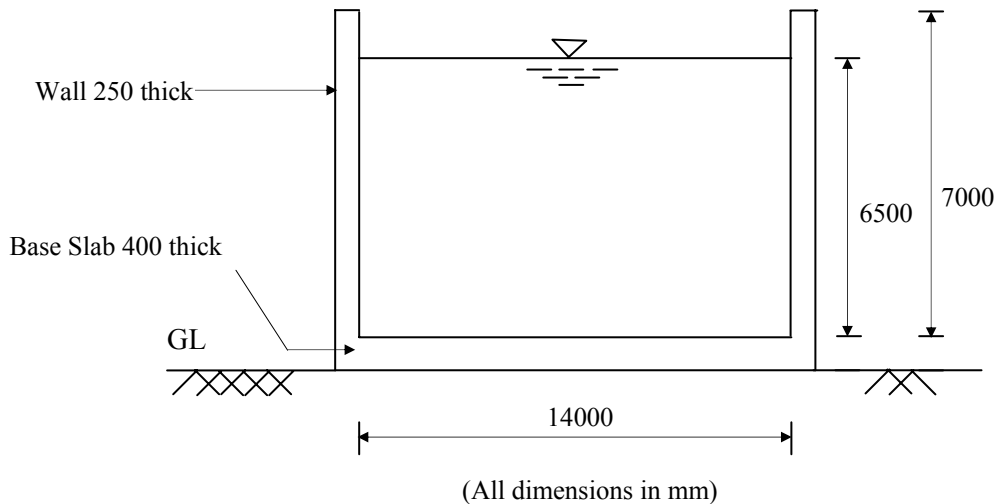


Figure 5.1 Sectional elevation

Solution:

5.1. Weight Calculations

Weight of tank wall

$$= \pi \times (14 + 0.25) \times 0.25 \times 25 \times 7.0$$

$$= 1,959 \text{ kN}$$

Mass of tank wall, m_w

$$= 1,959 \times 1,000 / 9.81$$

$$= 1,99,694 \text{ kg}$$

Mass of base slab, m_b

$$= \pi \times (7.25)^2 \times 0.4 \times 25 \times 1,000 / 9.81$$

$$= 1,68,328 \text{ kg.}$$

Volume of water = 1,000 m³

Mass of water, $m = 10,00,000 \text{ kg}$

Weight of water = 9,810 kN

5.2. Parameters of Spring Mass Model

$h = 6.5 \text{ m}; D = 14 \text{ m}$

For $h / D = 6.5 / 14 = 0.46$,

$$m_i / m = 0.511;$$

$$m_i = 0.511 \times 10,00,000 = 5,11,000 \text{ kg}$$

$$m_c / m = 0.464;$$

$$m_c = 0.464 \times 10,00,000 = 4,64,000 \text{ kg}$$

$$h_i / h = 0.375; \quad h_i = 0.375 \times 6.5 = 2.44 \text{ m}$$

$$h_c / h = 0.593; \quad h_c = 0.593 \times 6.5 = 3.86 \text{ m}$$

$$h_i^* / h = 0.853; \quad h_i^* = 0.853 \times 6.5 = 5.55 \text{ m}$$

$$h_c^* / h = 0.82; \quad h_c^* = 0.82 \times 6.5 = 5.33 \text{ m}$$

(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/m³,

t = Thickness of wall = 0.25 m,

D = Inner diameter of tank = 14 m ,

E = Young's modulus

$$= 5,000 \sqrt{f_{ck}}$$

$$= 5,000 \times \sqrt{30}$$

$$= 27,390 \text{ N/mm}^2$$

$$= 27,390 \times 10^6 \text{ N/m}^2.$$

For $h/D = 0.46$, $C_i = 4.38$

(Section 4.3.1.1)

$$T_i = \frac{4.38 \times 6.5 \times \sqrt{1,000}}{\sqrt{(0.25/14) \times \sqrt{27,390 \times 10^6}}}$$

$$= 0.04 \text{ sec.}$$

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)

$$= 0.225 \times (5,11,000 + 1,99,694 + 0) \times 9.81$$

$$= 1,569 \text{ kN}$$

Similarly, base shear in convective mode,

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

(Section 4.6.1)

$$= 0.065 \times 4,64,000 \times 9.81$$

$$= 296 \text{ kN}$$

Total base shear at the bottom of wall,

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

(Section 4.6.3)

$$= \sqrt{(1,569)^2 + (296)^2}$$

$$= 1,597 \text{ kN.}$$

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,

$$M_i = (A_h)_i [m_i h_i + m_w h_w + m_t h_t] g \quad (\text{Section 4.7.1.1})$$

$$= 0.225 \times [(5,11,000 \times 2.44) + (1,99,694 \times 3.5) + 0] \times 9.81$$

$$= 4,295 \text{ kN-m}$$

Similarly, bending moment in convective mode,

$$M_c = (A_h)_c m_c h_c g \quad (\text{Section 4.7.1.1})$$

$$= 0.065 \times 4,64,000 \times 3.86 \times 9.81$$

$$= 1,142 \text{ kN-m}$$

Total bending moment at bottom of wall,

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

$$= \sqrt{(4,295)^2 + (1,142)^2}$$

$$= 4,444 \text{ kN-m.}$$

5.7. Overturning Moment

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

$$M_i^* = (A_h)_i [m_i (h_i^* + t_b) + m_w (h_w + t_b) + m_t (h_t + t_b) + m_b t_b / 2] g \quad (\text{Section 4.7.1.2})$$

$$= 0.225 \times [(5,11,000 \times (5.55 + 0.4)) + (1,99,694 \times (3.5 + 0.4)) + 0 + (1,68,328 \times 0.4 / 2)] \times 9.81$$

$$= 8,504 \text{ kN-m.}$$

Similarly, overturning moment in convective mode,

$$M_c^* = (A_h)_c m_c (h_c^* + t_b) g \quad (\text{Section 4.7.1.2})$$

$$= 0.065 \times 4,64,000 \times (5.33 + 0.4) \times 9.81$$

$$= 1,695 \text{ kN-m.}$$

Total overturning moment at the bottom of base slab,

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

$$= \sqrt{(8,504)^2 + (1,695)^2}$$

$$= 8,671 \text{ kN-m.}$$

5.8. Sloshing Wave Height

Maximum sloshing wave height,

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

$$= 0.065 \times 2.0 \times 14 / 2$$

$$= 0.91 \text{ m}$$

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.

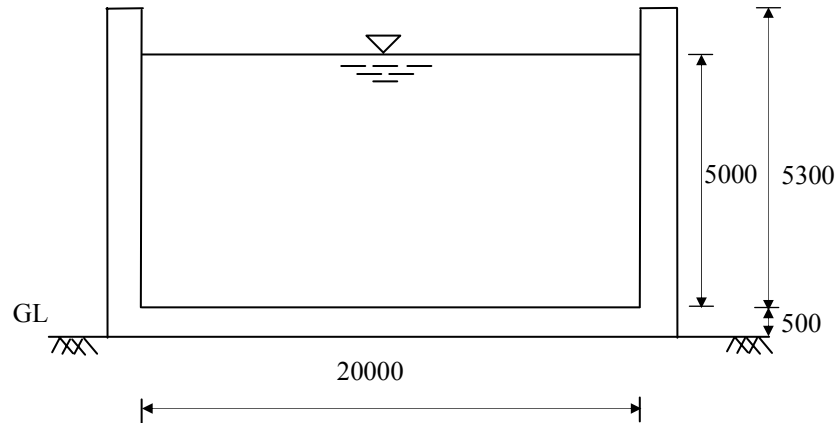
(Section 4.12)

Hydrodynamic pressure calculations for this tank are not shown. These will be similar to those in Example 4.

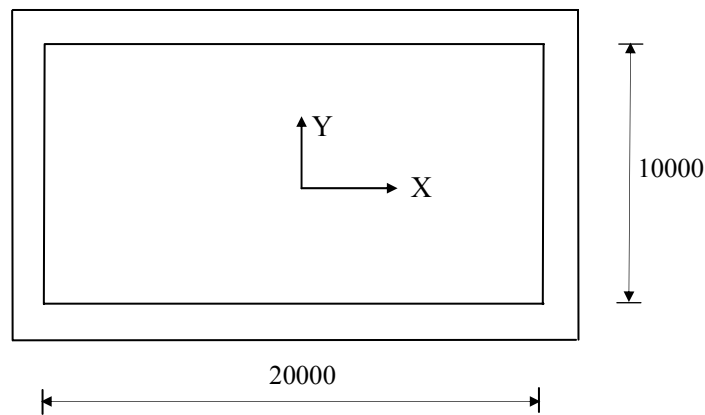
Example 6: Ground Supported Rectangular Concrete Tank

6. Problem Statement:

A ground supported rectangular RC water tank of 1,000 m³ 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.



(a) Elevation



(b) Plan

(All Dimensions in mm)

Figure 6.1 Details of tank geometry

6.1. Weight Calculations

Weight of tank wall

$$= 2 \times (20.4 + 10.4) \times 0.4 \times 25 \times 5.3$$

$$= 3,265 \text{ kN}$$

Mass of tank wall, m_w

$$= 3,265 \times 1,000 / 9.81$$

$$= 3,32,824 \text{ kg.}$$

Mass of base slab, m_b

$$= 10.8 \times 20.8 \times 0.5 \times 25 \times 1,000 / 9.81$$

$$= 2,86,239 \text{ kg.}$$

Volume of water = 1,000 m³

Weight of water = 10 x 20 x 5 x 9.81 = 9,810 kN

Mass of water, $m = 10,00,000 \text{ kg}$

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 $h/L = 5/20 = 0.25$,

$$m_i/m = 0.288;$$

$$m_i = 0.288 \times 10,00,000 = 2,88,000 \text{ kg}$$

$$m_c/m = 0.695;$$

$$m_c = 0.695 \times 10,00,000 = 6,95,000 \text{ kg}$$

$$h_i/h = 0.375 \quad ; \quad h_i = 0.375 \times 5 = 1.88 \text{ m}$$

$$h_c/h = 0.524 \quad ; \quad h_c = 0.524 \times 5 = 2.62 \text{ m}$$

$$h_i^*/h = 1.61 \quad ; \quad h_i^* = 1.61 \times 5 = 8.05 \text{ m}$$

$$h_c^*/h = 2.0 \quad ; \quad h_c^* = 2.0 \times 5 = 10.0 \text{ m.}$$

(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,

$$T_i = 2\pi \sqrt{\frac{d}{g}} \quad (\text{Section 4.3.1.2})$$

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 ,

(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.

$$\begin{aligned} &= 5.3 \times 0.4 \times 10 \times 25 \times 1,000 / 9.81 \\ &= 54,027 \text{ kg} \end{aligned}$$

Hence,

$$\bar{h} = \frac{\frac{2,88,000}{2} \times 1.88 + 54,027 \times \frac{5.3}{2}}{\frac{2,88,000}{2} + 54,027} = 2.09 \text{ m}$$

$$\begin{aligned} q &= \frac{\left(\frac{m_i}{2} + \bar{m}_w\right) g}{B h} \\ &= \frac{\left(\frac{2,88,000}{2} + 54,027\right) \times 9.81}{10 \times 5} \\ &= 38.9 \text{ kN/m}^2 \end{aligned}$$

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 $P = q \times h \times 1 = 38.9 \times 5 \times 1 = 194.5$ kN. Length of the cantilever is \bar{h} . Hence,

$$d = \frac{P(\bar{h})^3}{3 E I_w}$$

Where,

$$\begin{aligned} E &= 5,000 \sqrt{f_{ck}} = 5,000 \times \sqrt{30} \\ &= 27,390 \text{ N/mm}^2 \\ &= 27.39 \times 10^6 \text{ kN/m}^2 \end{aligned}$$

I_w = Moment of inertia of cantilever

$$= 1.0 \times \frac{t^3}{12} = 1.0 \times \frac{0.4^3}{12} = 5.33 \times 10^{-3} \text{ m}^4$$

Hence,

$$d = \frac{194.5 \times 2.09^3}{3 \times 27.39 \times 10^6 \times 5.33 \times 10^{-3}} = 0.00405 \text{ m}$$

$$T_i = 2\pi \sqrt{\frac{0.00405}{9.81}} = 0.13 \text{ sec.}$$

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)

$$= 0.34 \times (2,88,000 + 3,32,824 + 0) \times 9.81$$

$$= 2,071 \text{ kN.}$$

Similarly, base shear in convective mode,

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

(Section 4.6.1)

$$= 0.038 \times 6,95,000 \times 9.81$$

$$= 259 \text{ kN}$$

Total base shear at the bottom of wall,

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

(Section 4.6.3)

$$= \sqrt{(2,071)^2 + (259)^2}$$

$$= 2,087 \text{ kN.}$$

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)

$$= 0.34 \times [(2,88,000 \times 1.88) +$$

$$(3,32,824 \times 2.65) + 0] \times 9.81$$

$$= 4,747 \text{ kN-m}$$

Similarly, bending moment in convective mode,

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

(Section 4.7.1.1)

$$= 0.038 \times 6,95,000 \times 2.62 \times 9.81$$

$$= 679 \text{ kN-m}$$

Total bending moment at bottom of wall,

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

$$= \sqrt{(4,747)^2 + (679)^2}$$

$$= 4,795 \text{ kN-m.}$$

6.2.6. Overturning Moment

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

$$M_i^* = (A_h)_i [m_i(h_i^* + t_b) + m_w(h_w + t_b) + m_t(h_t + t_b) + m_b t_b / 2] g \quad (\text{Section 4.7.1.2})$$

$$= 0.34 \times [(2,88,000 \times (8.05 + 0.5) + (3,32,824 \times (2.65 + 0.5) + 0 + (2,86,239 \times 0.5 / 2))] \times 9.81$$

$$= 11,948 \text{ kN-m.}$$

Similarly, overturning moment in convective mode,

$$M_c^* = (A_h)_c m_c (h_c^* + t_b) g \quad (\text{Section 4.7.1.2})$$

$$= 0.038 \times 6,95,000 \times (10 + 0.5) \times 9.81$$

$$= 2,721 \text{ kN-m.}$$

Total overturning moment at the bottom of base slab,

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

$$= \sqrt{(11,948)^2 + (2,721)^2}$$

$$= 12,254 \text{ kN-m.}$$

6.2.7. Hydrodynamic Pressure

6.2.7.1. Impulsive Hydrodynamic Pressure

Impulsive hydrodynamic pressure on wall is

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

$$Q_{iw}(y) = 0.866 [1 - (y/h)^2] \times \tanh(0.866 L/h) \quad (\text{Section 4.9.1.(b)})$$

At base of wall, $y = 0$;

$$Q_{iw}(y = 0) = 0.866 [1 - (0/5)^2] \times \tanh(0.866 \times 20/5)$$

$$= 0.86.$$

Impulsive pressure at the base of wall,

$$p_{iw}(y = 0) = 0.86 \times 0.34 \times 1,000 \times 9.81 \times 5$$

$$= 14.3 \text{ kN/m}^2.$$

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

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

$$Q_{ib}(x) = \sinh(0.866 x/L) / \cosh(0.866 L/h) \quad (\text{Section 4.9.1(a)})$$

$$= \sinh(0.866 \times 20/10) / \cosh(0.866 \times 20/5)$$

$$= 0.171$$

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

$$p_{ib} = 0.171 \times 0.34 \times 1,000 \times 9.81 \times 5$$

$$= 2.9 \text{ kN/m}^2$$

6.2.7.2. Convective Hydrodynamic Pressure

Convective hydrodynamic pressure on wall is

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

$$Q_{cw}(y) = 0.4165 \frac{\cosh\left(3.162 \frac{y}{L}\right)}{\cosh\left(3.162 \frac{h}{L}\right)} \quad (\text{Section 4.9.2.(b)})$$

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,

$$p_{cw}(y = 0) = 0.31 \times 0.038 \times 1,000 \times 9.81 \times 20$$

$$= 2.31 \text{ kN/m}^2$$

At $y = h$;

$$Q_{cw}(y = h) = 0.4165$$

Convective pressure at $y = h$,

$$p_{cw}(y = h) = 0.4165 \times 0.038 \times 1,000 \times 9.81 \times 20$$

$$= 3.11 \text{ kN/m}^2$$

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

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

$$Q_{cb}(x) = 1.25 [x/L - 4/3 (x/L)^3] \text{sech}(3.162 h/L) \quad (\text{Section 4.9.2(a)})$$

$$= 1.25[L/2L - 4/3 (L/2L)^3] \operatorname{sech} \left(\frac{3.162 \times 5}{20} \right)$$

$$= 0.313$$

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,

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

$$= 0.34 \times 0.4 \times 25$$

$$= 3.4 \text{ kN/m}^2$$

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,

$$p_v = (A_v) [\rho g h (1 - y/h)] \quad (\text{Section 4.10.1})$$

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

$$Z = 0.36 \quad (\text{IS 1893(Part 1): Table 2; Zone V})$$

$$I = 1.5 \quad (\text{Table 1})$$

$$R = 2.0$$

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,

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}^2 + p_v^2} \quad (\text{Section 4.10.2})$$

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,

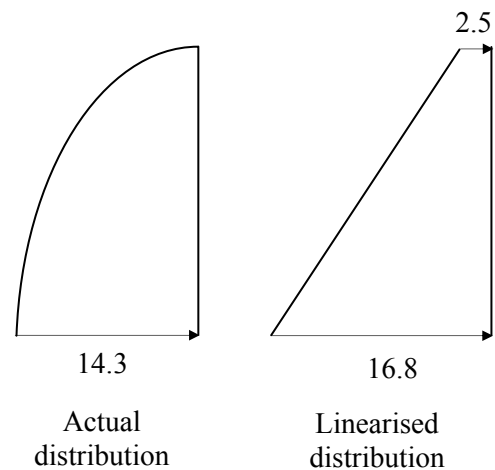
$$a_i = \frac{q_i}{h^2} (4h - 6h_i) = \frac{48.03}{5^2} (4 \times 5 - 6 \times 1.88)$$

$$= 16.8 \text{ kN/m}^2$$

$$b_i = \frac{q_i}{h^2} (6h_i - 2h) = \frac{48.03}{5^2} (6 \times 1.88 - 2 \times 5)$$

$$= 2.5 \text{ kN/m}^2$$

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 ,

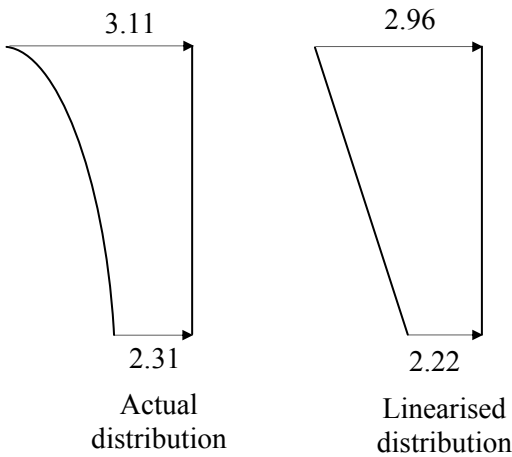
$$q_c = \frac{(A_h)_c m_c g}{2B} = \frac{0.038 \times 6,95,000 \times 9.81}{2 \times 10} = 12.95 \text{ kN/m}$$

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:



6.2.12. Sloshing Wave Height

Maximum sloshing wave height,

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

$$= 0.038 \times 2.0 \times 20 / 2$$

$$= 0.76 \text{ m}$$

Sloshing wave height is more than free board of 0.5 m.

6.2.13. Anchorage Requirement

$$\text{Here, } \frac{h}{L} = \frac{5}{20} = 0.25 ; \frac{1}{(A_h)_i} = \frac{1}{0.34} = 2.94$$

$$\text{As } \frac{h}{L} < \frac{1}{(A_h)_i}$$

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, $L = 10 \text{ m}$ and $B = 20 \text{ m}$.

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, $h/L = 0.5$, $m_i / m = 0.542$;

$$m_i = 0.542 \times 10,00,000 = 5,42,000 \text{ kg}$$

$$m_c / m = 0.485;$$

$$m_c = 0.485 \times 10,00,000 = 4,85,000 \text{ kg}$$

$$h_i / h = 0.375 ; h_i = 0.375 \times 5 = 1.88 \text{ m}$$

$$h_c / h = 0.583 ; h_c = 0.583 \times 5 = 2.92 \text{ m}$$

$$h_i^* / h = 0.797 ; h_i^* = 0.797 \times 5 = 4.0 \text{ m}$$

$$h_c^* / h = 0.86 ; h_c^* = 0.86 \times 5 = 4.3 \text{ m}$$

(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,

$$T_i = 2\pi \sqrt{\frac{d}{g}} \quad (\text{Section 4.3.1.2})$$

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 ,

$$\bar{h} = \frac{\frac{m_i}{2} h_i + \frac{m_w}{2} \bar{h}}{\frac{m_i}{2} + \frac{m_w}{2}}$$

\bar{m}_w = mass of one tank wall perpendicular to direction of loading.

$$= 5.3 \times 0.4 \times 20 \times 25 \times 1,000 / 9.81$$

$$= 1,08,053 \text{ kg}$$

$$\bar{h} = \frac{\frac{5,42,000}{2} \times 1.88 + 1,08,053 \times \frac{5.3}{2}}{\frac{5,42,000}{2} + 1,08,053}$$

$$= 2.1 \text{ m.}$$

$$q = \frac{\left(\frac{m_i}{2} + m_w\right)g}{Bh}$$

$$= \frac{\left(\frac{5,42,000}{2} + 1,08,053\right) \times 9.81}{20 \times 5}$$

$$= 37.2 \text{ kN/m}^2$$

Hence, $P = 37.2 \times 1 \times 5 = 186 \text{ kN}$.

As explained in Section 6.2.2 of this example,

$$d = \frac{P(\bar{h})^3}{3EI_w}$$

Where,

$$E = 27.39 \times 10^6 \text{ kN/m}^2,$$

$$I_w = 1.0 \times \frac{t^3}{12} = 1.0 \times \frac{0.4^3}{12} = 5.33 \times 10^{-3} \text{ m}^4$$

$$d = \frac{186 \times 2.1^3}{3 \times 27.39 \times 10^6 \times 5.33 \times 10^{-3}} = 0.00393 \text{ m}$$

$$\text{Hence, } T_i = 2\pi \sqrt{\frac{0.00393}{9.81}} = 0.13 \text{ sec}$$

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,

$$Z = 0.36 \quad (\text{IS 1893(Part 1): Table 2; Zone V})$$

$$I = 1.5 \quad (\text{Table 1})$$

Since this RC tank is fixed at base, R is taken as 2.0. (Table 2)

Here, $T_i = 0.13 \text{ sec}$,

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,

$$Z = 0.36 \quad (\text{IS 1893(Part 1): Table 2; Zone V})$$

$$I = 1.5 \quad (\text{Table 1})$$

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

Here, $T_c = 3.73 \text{ sec}$,

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)

$$= 0.34 \times (5,42,000 + 3,32,824 + 0) \times 9.81$$

$$= 2,918 \text{ kN}$$

Similarly, base shear in convective mode,

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

(Section 4.6.1)

$$= 0.06 \times 4,85,000 \times 9.81$$

$$= 300 \text{ kN}$$

Total base shear at the bottom of wall,

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

$$= \sqrt{(2,918)^2 + (300)^2}$$

$$= 2,933 \text{ kN.}$$

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,

$$M_i = (A_h)_i [m_i h_i + m_w h_w + m_t h_t] g \quad (\text{Section 4.7.1.1})$$

$$= 0.34 \times [(5,42,000 \times 1.88) + (3,32,824 \times 2.65) + 0] \times 9.81$$

$$= 6,340 \text{ kN-m}$$

Similarly, bending moment in convective mode,

$$M_c = (A_h)_c m_c h_c g \quad (\text{Section 4.7.1.1})$$

$$M_c = (A_h)_c m_c h_c g = 0.06 \times 4,85,000 \times 2.92 \times 9.81$$

$$= 875 \text{ kN-m}$$

Total bending moment at the bottom of wall,

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

$$= \sqrt{(6,340)^2 + (875)^2}$$

$$= 6,400 \text{ kN-m.}$$

6.3.6. Overturning Moment

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

$$M_i^* = (A_h)_i [m_i (h_i^* + t_b) + m_w (h_w + t_b) + m_t (h_t + t_b) + m_b t_b / 2] g \quad (\text{Section 4.7.1.2})$$

$$= 0.34 \times [(5,42,000 \times (4.0 + 0.5)) + (3,32,824 \times (2.65 + 0.5)) + 0 + (2,86,239 \times 0.5 / 2)] \times 9.81$$

$$= 11,870 \text{ kN-m.}$$

Similarly, overturning moment in convective mode,

$$M_c^* = (A_h)_c m_c (h_c^* + t_b) g \quad (\text{Section 4.7.1.2})$$

$$= 0.06 \times 4,85,000 \times (4.3 + 0.5) \times 9.81$$

$$= 1,439 \text{ kN-m.}$$

Total overturning moment at the bottom of base slab,

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

$$= \sqrt{(11,870)^2 + (1,439)^2}$$

$$= 11,957 \text{ kN-m.}$$

6.3.7. Hydrodynamic Pressure

6.3.7.1. Impulsive Hydrodynamic Pressure

Impulsive hydrodynamic pressure on wall is

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

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

(Section 4.9.1.(b))

At base of wall, $y = 0$;

$$Q_{iw}(y = 0) = 0.866 [1 - (0/5)^2] \times \tanh(0.866 \times 10/5) = 0.81$$

Impulsive pressure at the base of wall,

$$p_{iw}(y = 0) = 0.81 \times 0.34 \times 1,000 \times 9.81 \times 5 = 13.5 \text{ kN/m}^2.$$

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))

$$= \sinh(0.866 \times 10/10) / \cosh(0.866 \times 10/5)$$

$$= 0.336$$

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

$$p_{ib} = 0.336 \times 0.34 \times 1,000 \times 9.81 \times 5 = 5.6 \text{ kN/m}^2$$

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,

$$p_{cw}(y = 0) = 0.16 \times 0.06 \times 1,000 \times 9.81 \times 10$$

$$= 1.0 \text{ kN/m}^2$$

At $y = h$;

$$Q_{cw}(y = h) = 0.4165$$

Convective pressure at the $y = h$;

$$p_{cw}(y = h) = 0.4165 \times 0.06 \times 1,000 \times 9.81 \times 10$$

$$= 2.57 \text{ kN/m}^2.$$

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

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

$$Q_{cb}(x) = 1.25[x/L - 4/3 (x/L)^3] \text{ sech} (3.162 h/L)$$

(Section 4.9.2(a))

$$= 1.25[L/2L - 4/3 (L/2L)^3] \text{ sech} (3.162 \times 5/10)$$

$$= 0.165$$

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

$$p_{cb} = 0.165 \times 0.06 \times 1,000 \times 9.81 \times 10$$

$$= 1.02 \text{ kN/m}^2$$

6.3.8. Pressure Due to Wall Inertia

Pressure on wall due to its inertia,

$$p_{ww} = (A_h)_i t \rho_m g \quad (\text{Section 4.9.3})$$

$$= 0.34 \times 0.4 \times 25$$

$$= 3.4 \text{ kN/m}^2.$$

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,

$$p_v = (A_v) [\rho g h (1 - y/h)] \quad (\text{Section 4.10.1})$$

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

$$Z = 0.36 \quad (\text{IS 1893(Part 1): Table 2; Zone V})$$

$$I = 1.5 \quad (\text{Table 1})$$

$$R = 2.0$$

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., $y = 0$,

$$p_v = 0.225 \times [1 \times 9.81 \times 5 \times (1 - 0/5)]$$

$$= 11.04 \text{ kN/m}^2$$

6.3.10. Maximum Hydrodynamic Pressure

Maximum hydrodynamic pressure,

$$p = \sqrt{(p_{iw} + p_{ww})^2 + p_{cw}^2 + p_v^2} \quad (\text{Section 4.10.2})$$

At the base of wall,

$$p = \sqrt{(13.5 + 3.4)^2 + 1.0^2 + 11.04^2}$$

$$= 20.22 \text{ kN/m}^2.$$

This maximum hydrodynamic pressure is about 41 % of hydrostatic pressure (49 kN/m²). This being more than 33%, design of tank will be influenced by hydrodynamic pressure.

6.3.11. Sloshing Wave Height

Maximum sloshing wave height,

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

$$= 0.06 \times 2.0 \times 10 / 2$$

$$= 0.63 \text{ m}$$

6.3.12. Anchorage Requirement

Here, $\frac{h}{L} = \frac{5}{10} = 0.5$;

$$\frac{1}{(A_h)_i} = \frac{1}{0.34} = 2.94$$

$$\text{As } \frac{h}{L} < \frac{1}{(A_h)_i}$$

No anchorage is required.

(Section 4.12)

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ISBN 81-904190-4-8

A NICEE Publication

IITK-GSDMA GUIDELINES for

SEISMIC DESIGN of LIQUID STORAGE TANKS
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October 2007