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व्यापक परिचालन मसौदा

हमारा संदर्भ : सीईडी 38/टी-09 **13 नवंबर 2024**

तकनीकी समिति :विशेष संरचना विषय समिति, सीईडी 38

प्राप्तकर्ता :

क) सिविल अभियांत्रिकी विभाग परिषद,सीईडीसी के सभी सदस्य
ख) विशेष संरचना विषय समिति, सीईडी 38और इसकी उपसमितियों केसभी सदस्य
ग) रुचि रखने वाले अन्य निकाय।

प्रिय महोदय,महोदया/

निम्नलिखित भारतीय मानक का मसौदा संलग्न हैं:

प्रलेखसंख्या	शीर्षक
सीईडी 38(26928)WC	**उच्चतरल भंडारण टैंक के लिए प्रबलित कंक्रीट के डिज़ाइन के लिए मानदंड का भारतीय मानक मसौदा** (आईएस 11682 का *पहला पुनरीक्षण*) आईसीएस 91.080.40

कृपया इस मसौदे का अवलोकन करें और अपनी सम्मतियाँ यह बताते हुए भेजे कि यह मसौदा प्रकाशित हो तो इन पर अमल करने में आपको व्यवसाय अथवा कारोबार में क्या कठिनाइयां आ सकती हैं।

सम्मतियाँ भेजने की अंतिम तिथि: **29 दिसंबर 2024**

सम्मति यदि कोई हो तो कृपया अधोहस्ताक्षरी को ई-मेल द्वारा ced38@bis.gov.in पर या उपरलिखित पते पर, संलग्न फोर्मेट में भेजें। सम्मतियाँ बीआईएस ईगवर्नेंस पोर्टल- www.manakonline.in के माध्यम से ऑनलाइन भी भेजी जा सकती हैं।

यदि कोई सम्मति प्राप्त नहीं होती है अथवा सम्मति में केवल भाषा संबंधी त्रुटि हुई तो उपरोक्त प्रालेख को यथावत अंतिम रूप दे दिया जाएगा। यदि सम्मति तकनीकी प्रकृति की हुई तो विषय समिति के अध्यक्ष के परामर्श से अथवा उनकी इच्छा पर आगे की कार्यवाही के लिए विषय समिति को भेजे जाने के बाद प्रालेख को अंतिम रूप दे दिया जाएगा।

यह प्रालेख भारतीय मानक ब्यूरो की वेबसाइट www.bis.gov.in पर भी उपलब्ध हैं।

धन्यवाद।

भवदीय
ह-/
(द्वैपायन भद्र)
वैज्ञानिक ई एवं प्रमुख
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WIDE CIRCULATION DRAFT

Our Reference: CED 38/T-09 **13 November 2024**

TECHNICAL COMMITTEE: Special Structures Sectional Committee, CED 38

ADDRESSED TO:

a) All Members of Civil Engineering Division Council, CEDC
b) All Members of Special Structures Sectional Committee, CED 38 and its Subcommittees
c) All others interested.

Dear Sir/Madam,

Please find enclosed the following draft:

Doc No.	Title
CED 38(26928)WC	**Draft Indian Standard** **Criteria for Design of Reinforced Concrete Staging for Elevated Liquid Storage Tanks** (*First Revision* of IS 11682) ICS 91.080.40

Kindly examine the attached draft and forward your views stating any difficulties which you are likely to experience in your business or profession, if this is finally adopted as National Standard.

Last Date for comments**: 29 December 2024**

Comments if any, may please be made in the enclosed format and emailed at ced38@bis.gov.in or sent at the above address. Additionally, comments may be sent online through the BIS e-governance portal, www.manakonline.in.

In case no comments are received or comments received are of editorial nature, kindly permit us to presume your approval for the above document as finalized. However, in case comments, technical in nature are received, then it may be finalized either in consultation with the Chairman, Sectional Committee or referred to the Sectional Committee for further necessary action if so desired by the Chairman, Sectional Committee.

The document is also hosted on BIS website www.bis.gov.in.

Thanking you,

Yours faithfully,

Sd/-

(Dwaipayan Bhadra)

Scientist 'E' & Head

Civil Engineering Department

Email:ced38@bis.gov.in

Encl: As above

FORMAT FOR SENDING COMMENTS ON BIS DOCUMENTS

(Please use A-4 size sheet of paper only and type within fields indicated. Comments on each clause/sub-clause/table/fig etc. be started on a fresh box. Information in column 3 should include reasons for the comments and suggestions for modified working of the clauses when the existing text is found not acceptable. Adherence to this format facilitates Secretariat's work) {Please e-mail your comments to ced38@bis.gov.in}

Doc. No.: **CED 38(26928)WC** **BIS Letter Ref**: CED 38/T-09

Title: Draft Indian Standard Criteria for Design of Reinforced Concrete Staging for Elevated Liquid Storage Tanks
(*First Revision* of IS 11682) ICS 91.080.40

LAST DATE OF COMMENTS: **29/12/2024**

NAME OF THE COMMENTATOR/ ORGANIZATION: ______________________________

Sl No.	Clause/ Para/ Table/ Figure No. commented	Type of Comment (General/ Technical/ Editorial)	Comments/ Modified Wordings	Justification of Proposed Change
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NOTE- Kindly insert more rows as necessary for each clause/table, etc

BUREAU OF INDIAN STANDARDS

DRAFT FOR COMMENTS ONLY

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Draft Indian Standard

CRITERIA FOR DESIGN OF REINFORCED CONCRETE STAGING FOR ELEVATED LIQUID STORAGE TANKS

(*First Revision* of IS 11682)
ICS 91.080.40

Special Structures Sectional Committee, CED 38	Last Date for Comments: 29 December 2024

FOREWORD

(*Formal Clauses will be added later*)

Elevated liquid storage tanks are important public utility general classified as industrial structures. The specifications, design and construction method in reinforced concrete are influenced by the prevailing construction practices, the physical properties of the material and the environmental conditions.

The design and construction of container for storage of liquids have been covered in the series of IS 3370 'Concrete structures for retaining aqueous liquids – Code of practice', which are as follows:

(Part 1): 2021 General requirements (*second revision*)
(Part 2): 2021 Plan and reinforced concrete (*second revision*)
(Part 3): 2021 Prestressed concrete (*first revision*)
(Part 4) Design Tables:
(Sec 1): 2021 Plates (*first revision*)
(Sec 2): 2021 Rectangular tanks (*first revision*)
(Sec 3): 2021 Circular tanks (*first revision*)

This standard lays down the principles of design and detailing for staging of elevated liquid storage tanks. All requirements of IS 456 : 2000 'Plain and reinforced concrete — Code of practice (*fourth revision*)', IS 3370 (Parts 1 and 2), IS 1893 (Part 2) : 2014 'Criteria for earthquake resistant design of structures: Part 2 Liquid retaining tanks (*fifth revision*)' and IS 13920 : 2016 'Ductile design and detailing of reinforced concrete structures subjected to earthquake forces - Code of practice (*first revision*)' in so far as these apply, and shall be deemed to form part of this standard except where otherwise laid down in this standard.

This standard was first published in 1985. The first revision was taken up to keep abreast with the rapid developments in design and construction fields, and to bring further modifications in the light of experience gained in design and construction of staging of elevated tanks. This revision incorporates a number of important changes.

Some of the significant changes incorporated are as follows:

a) All the design provisions, as per limit state method, have been updated and made comprehensive.
b) A clause related to documentation of all salient features of the work, engineering data and brief maintenance scheme of the work is introduced.
c) Detailed recommendations for analysis including $P - \Delta$ analysis have been included.
d) Provisions with respect to stability of the elevated tank have been introduced.
e) The sub-clause on loads has been enlarged to include detailed guidance on the various types of loadings; and liquid load has been defined separately.
f) Partial safety factors for loads and various load combinations, applicable to liquid retaining structures, have been duly revised.
g) This standard is drafted for common types of staging. Enough details may not be available for all other types of staging and possible configurations, for which designer is responsible for additional criterion for design. Variations in the types and configurations may be possible.
h) Analysis for circular shaft for bending moment due to ovalling and vertical stress due to horizontal (wind/earthquake) load on the staging has been given.
j) Provisions relating to eccentricity of shaft and container have been given in detail. and
k) Construction requirements for design to achieve high level of quality for better reliability have been given.

While, the common methods of design have been covered in this standard. Design of structures of special forms, or that in unusual circumstances should be left to the judgment of the design engineer, and in such cases special systems of design and construction may be permitted on production of satisfactory evidence regarding their adequacy and safety by analysis or test or by both.

In this standard it is assumed that the design of liquid storage tank and staging is entrusted to the qualified engineer, knowledgeable with the current engineering practices related to analysis and design of reinforced concrete, and the execution of work is carried out under the direction of an experienced supervisor.

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

a) Seismic design of liquid storage tanks, Final report document no. IITK-GSDMA-EQ-20-V2.0, National Information Centre of Earthquake Engineering (NICEE), Indian Institute of Technology Kanpur, Kanpur.
b) ACI 371R 16, Guide for the analysis, design, and construction of elevated concrete and composite steel-concrete water storage tanks, American Concrete Institute.

This standard also aims at satisfying some Sustainable Development Goals by United Nations, especially Goal 9 ‘Industry, innovation and infrastructure’, particularly its target **9.1**; and also to the Goal 11 ‘Sustainable cities and Communities’, particularly its target **11.5**.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Draft Indian Standard

CRITERIA FOR DESIGN OF REINFORCED CONCRETE STAGING FOR ELEVATED LIQUID STORAGE TANKS

(First Revision)

1 SCOPE

1.1 This standard lays down criteria for planning, analysis, design and construction of frame type with columns and shaft type reinforced concrete (RC) staging. Liquid storage tank (container) may consist of any material like RC, fibre concrete, ferrocement, steel, polyvinyl chloride (PVC), etc.

NOTE – 'Liquid storage tank' and 'Liquid container' are treated as synonymous terms. In place of 'liquid', 'water' may be used wherever appropriate by the user.

1.2 While the provisions of this standard refer to the staging for the storage of liquid, the recommendations are applicable mainly to aqueous liquid storage or containment. Additional requirements may be necessary for containment of liquids other than ordinary or plain water.

1.3 For the container portion, reference shall be made to all provisions given IS 3370 (Parts 1, 2, 3 and 4).

1.4 The requirements given in this standard are not applicable for staging in reinforced masonry or un-reinforced masonry, may it be in concrete block, stone or bricks.

1.5 In the guidelines given for planning, choosing a layout of tank and staging, the configurations are indicative and not normative. The designer may adopt a layout or configuration not conforming to the guidelines; however, in all cases the designer is responsible to design a safe and sound structure. Economy of a solution will also depend upon the experience and training of workers, experience of the agency of construction, the formwork materials and equipment available with the agency of construction, etc. However, all requirements for stability, soundness, durability, structural integrity, and strength shall confirm to provisions in this standard.

2 REFERENCES

The standards given in Annex A, contain provisions which, through reference in this standard, constitute provisions of this standard. At the time of approval of this publication, the editions indicated were valid. All standards are subject to revisions, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of these standards.

3 TERMINOLOGY

For the purpose of this standard, the definitions given in IS 3370 (Part 1) and the following shall apply.

3.1 Base of Structure — A level at which the horizontal earthquake ground motions are assumed to be imparted to the structure. This is below the ground level and is generally taken at foundation level.

3.2 Brace — In this standard, brace is a RCC beam, an essential part of moment resisting frame, basically resisting bending moments, shear force and other internal force actions, and also referred as tie beam.

3.3 Force Actions — These include bending moments, torsion, shear forces, direct tension or compression. Indirect actions can be due to imposed deformations or earthquake.

3.4 Framed (or Frame type) Staging — It consists an assemblage of columns and tie beams.

3.5 Height of Staging — It is the difference between the lowest supply level of the tank and the average ground level at the tank site.

3.6 Lateral-force Resisting System — It is the part of the structure composed of members assigned to resist lateral forces due to wind, earthquake effects, etc. It may consist of moment frames or shear walls, or combination of both.

3.7 Liquid Depth — This shall be the difference of levels between the full supply level (FSL) or working top liquid level (WTL) of the tank, and the lowest supply level (LSL). In case of liquid being water, the term ‘water depth’ can be used. The ‘design liquid depth’ for tank can be more than the ‘liquid depth’ due to dead storage and due to rise of liquid in freeboard zone to be accounted for in design.

3.8 Moment Frame — Assemblage of columns and beams and their junctions that resists force mainly through flexure with shear, axial force, etc. It can form the lateral-force resisting system.

3.9 Plastic Hinge Region — Length of a frame element over which intended flexural yielding may occur due to earthquake load, and is assumed to extend not less than the depth of member from where flexural yielding initiates.

3.10 Shaft (or shaft type) Staging — It consists of a shell, like a circular or polygonal cylinder or hollow prism.

3.11 Staging — Staging is an assembly of components to support the elevated liquid storage tank significantly above ground level. Pedestals or blocks of short heights supporting a tank will not be called as staging. In general, the term staging includes the structural components for foundations also.

3.12 Storey or Storey Height — It is the vertical height of staging or columns between two successive levels of braces and/or beams. The first storey above foundation will be a height from the top of structural (RC) foundation to the set of ground (or plinth) level braces.

4 SYMBOLS AND NOTATIONS

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

C_{f} – Force coefficient (for wind load)
DL – Dead load (in N or N/m² or N/m² or N/m³)
DL_{p} – Provisional dead load (in N or N/m or N/m² or N/m³
d – Column size (diameter or smaller size of section)
EL – Earthquake load (in N)
FL – Liquid (fluid) pressure or load (in N/m² or N/m³)
FSL – Full supply level (in m)
f_{ck} – Characteristic compressive strength of concrete cubes (in N/mm²
f_{y} – Characteristic tensile yield or proof stress of reinforcement, (in N/mm² or MPa)
GL – Ground level
H – Depth of liquid in tank (in m)
H_{t} – Total height of elevated tank above foundation (including container/ up to roof top) (in m)
h_{sc} – Unsupported height of column between braces/ beams (in m)
IL – Imposed load (in N or N/m²)
IL_{p} – Imposed load due to an operation or equipment (in N or N/m²)
IL_{s} – Imposed load due to storage (in N or N/m²)
K_{d}, K_{a}, K_{c} – Wind speed factors (refer IS 875-3)
LSL – Lowest supply level (in m)
MTL – Maximum top liquid level (in m)
$OMRF$ – Ordinary moment resisting frame not conforming to IS 13920 ductile detailing
$P-\Delta$ – Effect of vertical load with horizontal deflection (including secondary effect) resulting in increased bending moment in staging, which is a non-linear effect.
p_{d} – Design wind pressure [as per **7.2** of IS 875 (Part 3)]
Q_{c} – Stability index (*see* **E-2** of IS 456) or MPa)
R – Response reduction factor for earthquake design,
R_{c} – Radius of the centre line of shaft (in m)
$SMRF$ – Special moment resisting frame conforming to IS 13920 ductile Detailing
t_{s} – Thickness of shaft (in m or mm)
WL – Wind load (in N or N/m²)
WTL – Normal working top liquid level (in m)
ε – Roughness [*refer* to IS 875 (Part 3) (in mm)]

5 SPECIFICATIONS, DESIGN REPORT AND DRAWINGS

Elevated liquid storage tanks in reinforced concrete are normally constructed under design and built contracts. The designs are checked by owner organizations or proof consultants. Hence, all objective data should be clearly defined and for subjective decisions if required, solutions should be defined, along with the data in contract document.

Documentation (including design basis document) shall be prepared which shall contain all salient features of the work, engineering data and brief maintenance scheme of the work as given in **5.1** to **5.12**.

5.1 Brief data and features like description of liquid to be contained, capacity of tank (in m^3), height of free board (in mm), depth of liquid in tank from LSL to FSL (in m), and staging height (in m) be given. Salient levels, average GL, foundation level, LSL, FSL, etc. shall also be given.

5.2 Foundation investigation report and soil data, type of foundation, probable depth of foundation and net safe, net allowable and ultimate bearing pressure of founding strata shall be given. The positions of ground water table (highest and lowest) shall be given. Soil classification for earthquake design, and liquefaction potential shall also be given. Loss of soil strength due to heavy rains or leakages shall be taken in to account for safety of the structure.

5.3 Location of elevated tank (for example, polluted industrial area, sea front area, costal area, urban area, rural area, etc.), and purpose of storage of liquid (such as public water supply, fire-fighting, industrial) shall be given. The information on pollutants, salts, sulphates if any in air, soil and ground water, if of significance at the concerned location shall also be given.

5.4 Specifications of concrete and its grade/grades, type of cement to be used, limits of maximum and minimum cement content, grade of reinforcement bars.

5.4.1 Guidance on locations of laps shall be given. Table of lap length in reinforcement of different types of members for each size of bars (diameter wise) shall be given.

5.5 Salient features of structure and construction, method of construction, height of column lifts, height of each wall lift, and guidance on release of formwork shall be given. For specialized formwork requirement, the design and drawing of the formwork shall be given.

5.6 Clear cover of concrete on reinforcement bars for various members at different locations shall be given.

5.7 General locations, specifications and treatment of construction joints should be specified.

5.8 Special protective coating if being specified, and its brief specifications.

5.9 References of codes, standards, and guidance for construction taken into consideration, shall be given.

5.10 Design loads

Density of concrete, liquid, soil, masonry, etc; provisional loads of finishing, flooring, rendering, coating, lining, etc as applicable; loads of railing, parapets, masonry wall, etc imposed loads on roof, balcony, walkways, platform, etc earthquake zone, zone factor, response reduction factor, importance factor, critical damping factor, soil factor [*see* IS 1893 (Part 1)]; basic wind speed, k_1, k_2, k_3, k_4, K_a, K_d, K_c, terrain category [*see* IS 875 (Part 3)]; load of equipment if any; construction loads; blast or explosion load if to be considered, any other loads considered, shall be given.

5.10.1 Special precautions shall be taken for the safety of elevated tanks stagings in cyclonic coastal area.

5.11 Design report containing basis of design, method of structural analysis, structural configuration and the assumptions made, detailed computation of loads, structural analysis, design calculations with sizes of members and reinforcement, checks required as per codes.

5.12 Drawings shall have reinforcement detailing, instructions, junction detailing, brief specifications and notes. Detailing at construction joints should be specified on the drawing.

6 EXPOSURE CONDITION

6.1 Components of staging shall be treated as exposed to not less than 'moderate' environment as defined in IS 456. Components of container in concrete shall be treated as per IS 3370 (Part 1).

6.2 Owner or designer may decide for higher exposure condition based on the location of the tank. For staging in coastal area and in area of heavy air pollution, higher environmental exposure condition like 'severe' or 'very severe' could apply.

6.3 For foundations and its components (like piles, footing, column, ground brace) in contact with ground or soil, based on actual ground or sub-soil conditions, higher exposure condition may be considered for design.

6.4 For severe or higher exposure conditions, possible mechanisms which could bring about durability loss shall be assessed and accordingly design specifications (concrete grade, cement content, clear cover, etc.) shall be applicable. Applications of coating or lining and precautions in construction shall be considered to achieve the designed service life of structure as applicable.

6.5 While deciding the design exposure condition, the possibility of slight seepage through container due to construction error may be considered, which would make the components of staging occasionally wet, and thus may need higher exposure condition.

7 CONCRETE

7.1 Cement

7.1.1 Cement type shall be as per IS 456.

7.1.1.1 Sulphate resisting Portland cement can be used only if environment has sulphates in significant quantity (*refer* to IS 456) and total chlorides are in very small quantity. Its use may be required for members below ground level. With sulphates, where chlorides are also in significant quantities, use of Portland slag cement will be appropriate; or ground granulated blast-furnace slag can be used as mineral admixture.

7.1.2 The minimum cement content shall confirm to the requirements of IS 456. The maximum cementitious (binder) content shall be as per IS 3370 (Part 1).

7.1.3 Use of blended cements [conforming to IS 1489 (Part 1), IS 455 and IS 16415] are preferable, unless 7 days strength of more than 20 N/mm² is the target.

7.1.4 Source (the manufacturing plant), grade and type of cement if changes during construction, mix proportioning shall be again verified by trial mix.

7.2 The requirements for other materials of concrete shall be governed by IS 456 for reinforced concrete, with the additional requirements given in **7.2.1** and **7.2.2**.

7.2.1 Use of aggregate having high porosity (more than 5 percent) shall be permitted only for staging in medium exposure condition, after establishing its parameters, long term influence on concrete and specifically effect on permeability and durability.

7.2.2 Prestressed members will be governed by IS 1343, if used. Structural steel members will be governed by IS 800, if used.

7.3 The maximum of the requirements in **7.3.1** to **7.3.7**, shall govern the concrete grade for staging.

7.3.1 The grade of concrete shall not be less than that required by IS 456, depending upon the exposure condition.

7.3.2 The grade of concrete shall not be less than M25 for all staging, except those specified in **7.3.4** and **7.3.5**.

7.3.3 The grade of concrete shall not be less than M30 for staging with any one of following conditions.

a) Tank in coastal areas or near sea face;
b) where basic wind speed > 50 m/s;
c) Tanks more than 1 000 m³;
d) Tanks more than 500 m³ in earthquake zone IV or more;
e) Tanks more than 500 m³ and staging height more than 20 m;
f) Tanks more than 200 m³ in earthquake zone V; and
g) Tanks more than 200 m³ and staging height more than 30 m.

7.3.4 For conformance to the requirement of 'maximum permitted water-cement ratio related to exposure condition and the maximum specified in IS 456, concrete grade required may be higher than that recommended above. In such a case, specified grade of concrete in construction should be not less than the developable strength at the limiting water-cement ratio specified.

7.3.4.1 Concrete of grades higher than that recommended, are acceptable as decided by owner or designer.

7.3.5 The grade of concrete for staging may preferably be same as that for container, for convenience in construction.

7.3.6 Ready mixed concrete (RMC) conforming to IS 4926 is acceptable.

7.3.7 Chemical admixtures complying with IS 9103 are permitted. Admixtures containing chlorides or $CaCl_2$ are not permitted, in view of risk of corrosion of steel reinforcement.

7.4 Fibres

For enhancing the performance of concrete, addition of fibres is permitted. In general steel or polymeric/synthetic fibres (conforming to IS 16481) can be added. For any other fibre, its long term chemical stability shall be established. Fibres control plastic shrinkage cracks, as well temperature-shrinkage cracks in young age of concrete. Structural fibres like polymeric or steel can improve the dispersion of cracks due to loads in service life, thereby reducing crack width and also enhance the toughness, ductility and durability. It may be noted that no strength enhancement due to use of fibres shall be considered in design.

7.5 Reinforcement SS steel as per IS 16651

7.5.1 Steel reinforcement shall conform to IS 1786 and IS 13920.

7.5.2 If specified, reinforcement can be stainless steel SS500 grade (as per IS 16651) with ratio of ultimate strength to actual 0.2 percent proof stress more than 1.12.

7.5.3 Welded plain or welded deformed wires or bars shall not be permitted at locations where reinforcement is governed by high ductility requirements as per IS 13920.

7.5.4 Nominal cover to reinforcement shall be governed by the exposure conditions assumed for design, and the design strategy for preventing loss of durability during service life. An additional margin for tolerance may also be considered for inclusion, as the staging is subjected to external exposure all the times.

7.5.5 Formwork shall comply with IS 456 and IS 14687.

8 STRUCTURAL CONFIGURATION

8.1 Top most level of staging shall be connected to the container preventing relative horizontal and vertical movements between members at top of staging in contact with container. The connection or junctions shall be designed to withstand the forces to which it may be subjected, and more specifically for tension, shear and bending during wind and earthquake actions.

For container in RC, monolithic connection between members of container and staging are preferred. If container is not of RC, there shall be arrangement for safe and efficient transfer of loads, forces, and actions between container and staging, including occasional uplift (due to horizontal loads), such that structural integrity is maintained.

8.2 In case of framed staging, all members carrying vertical loads shall be tied together at top as well as at bottom of staging (that is, near foundation). Staging member connected monolithically to container will not require additional tie members at top.

8.3 Bottoms of columns will be considered as tied if connected by

a) foundation beams or strip foundation; and
b) braces (tie beams) such that the clear distance between top of structural foundation and bottom of next brace (at or within ground) shall not be more than as specified in **14.5.2**.

Where plinth level is little above the GL, the GL brace can be partly above GL.

8.4 Different shapes of elevated tanks and their staging with certain arrangements of bottom construction are shown in figures. (*see* Figs.1 to 9).

8.5 Constructability, maintainability and restorability are also to be considered for design. Design should be such that construction is simple and easy, or guided by enough details to avoid shortfall in quality of construction. Design and details should also be such that the planned maintenance in future would be possible for retaining the serviceability of structure. In case of shortfall in quality, repairs and restoration should be conveniently possible.

8.6 General information on types of staging is given in Annex B, which is not exhaustive, and many other variations may be possible. Guidelines on the layout and configuration of staging are given in Annex C.

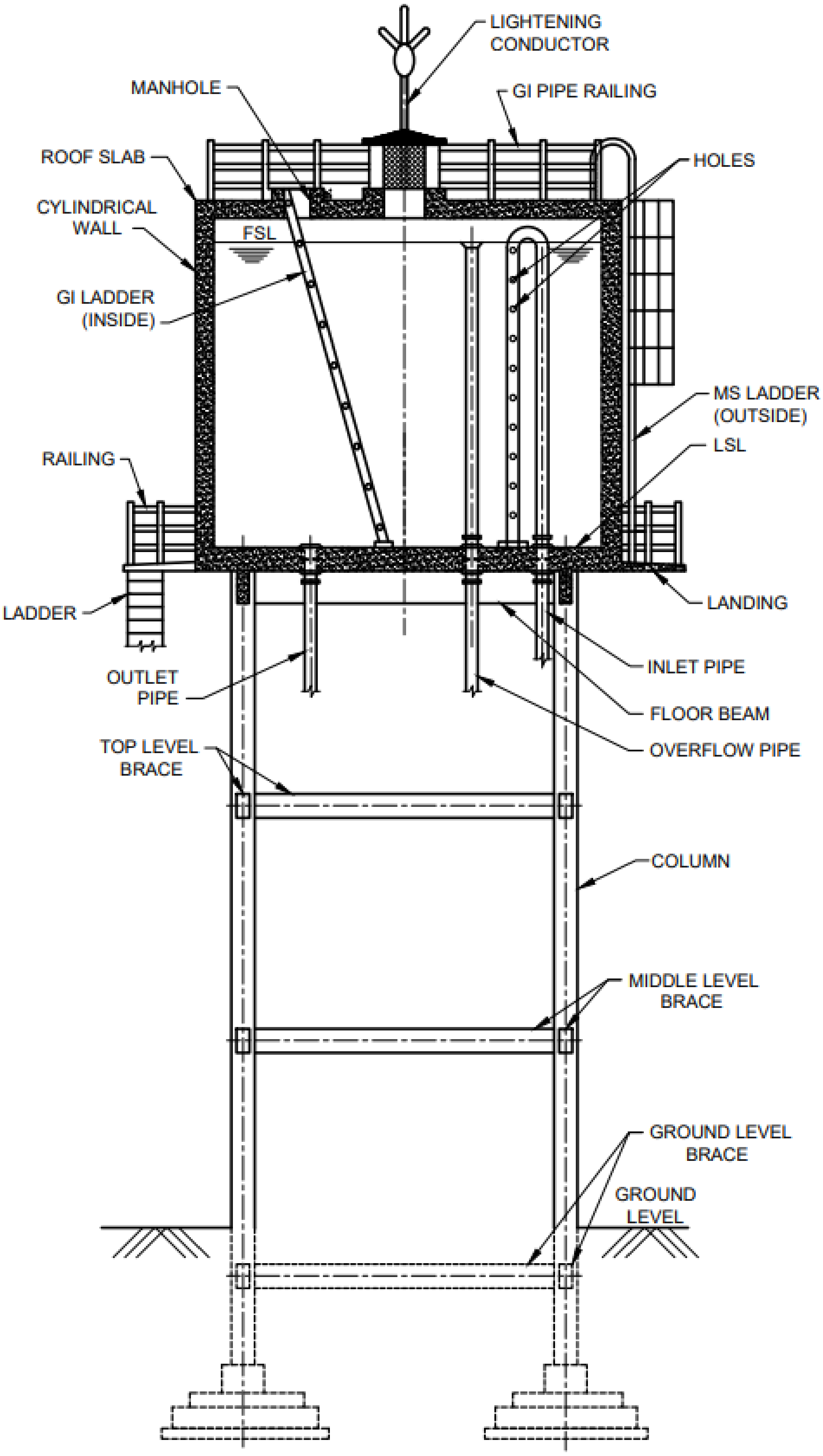


FIG. 1 TYPICAL SECTION OF ELEVATED CYLINDRICAL TANK (SMALL).

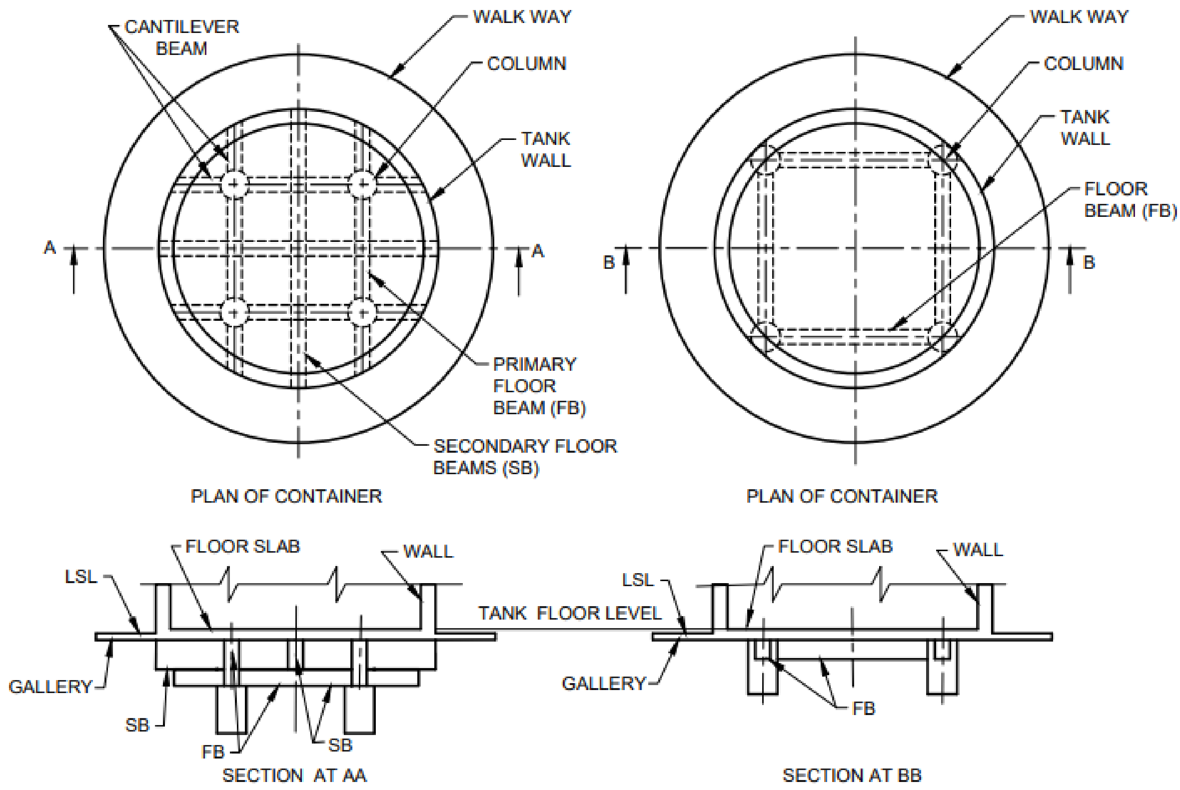


FIG. 2 TYPICAL ARRANGEMENT OF FLOOR SLAB FOR CYLINDRICAL TANKS ON FOUR COLUMNS.

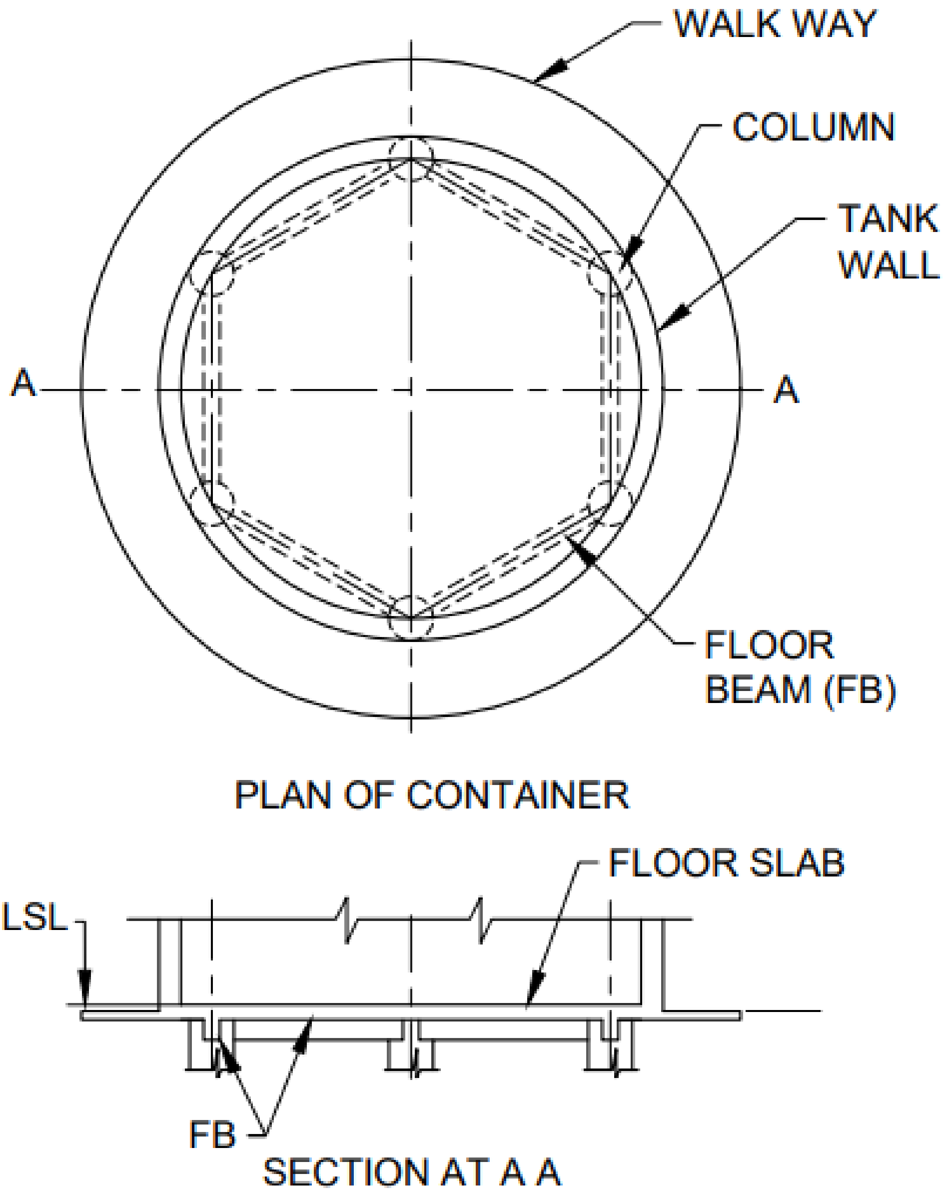


FIG. 3 TYPICAL ARRANGEMENTS OF FLOOR SLAB FOR CYLINDRICAL TANK ON SIX COLUMNS.

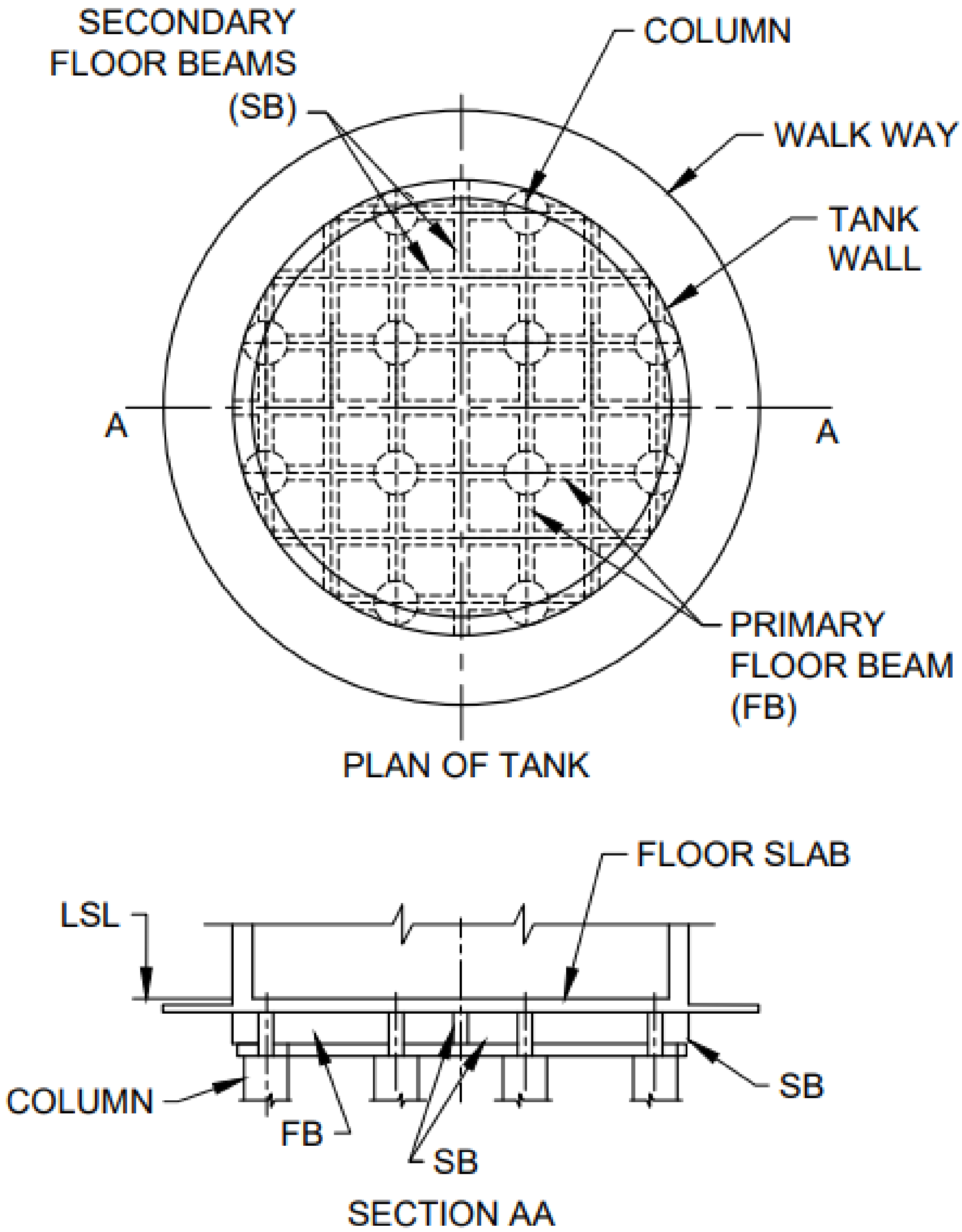


FIG. 4 FLOOR SLAB ON 12 COLUMNS STAGING WITH SECONDARY BEAMS.

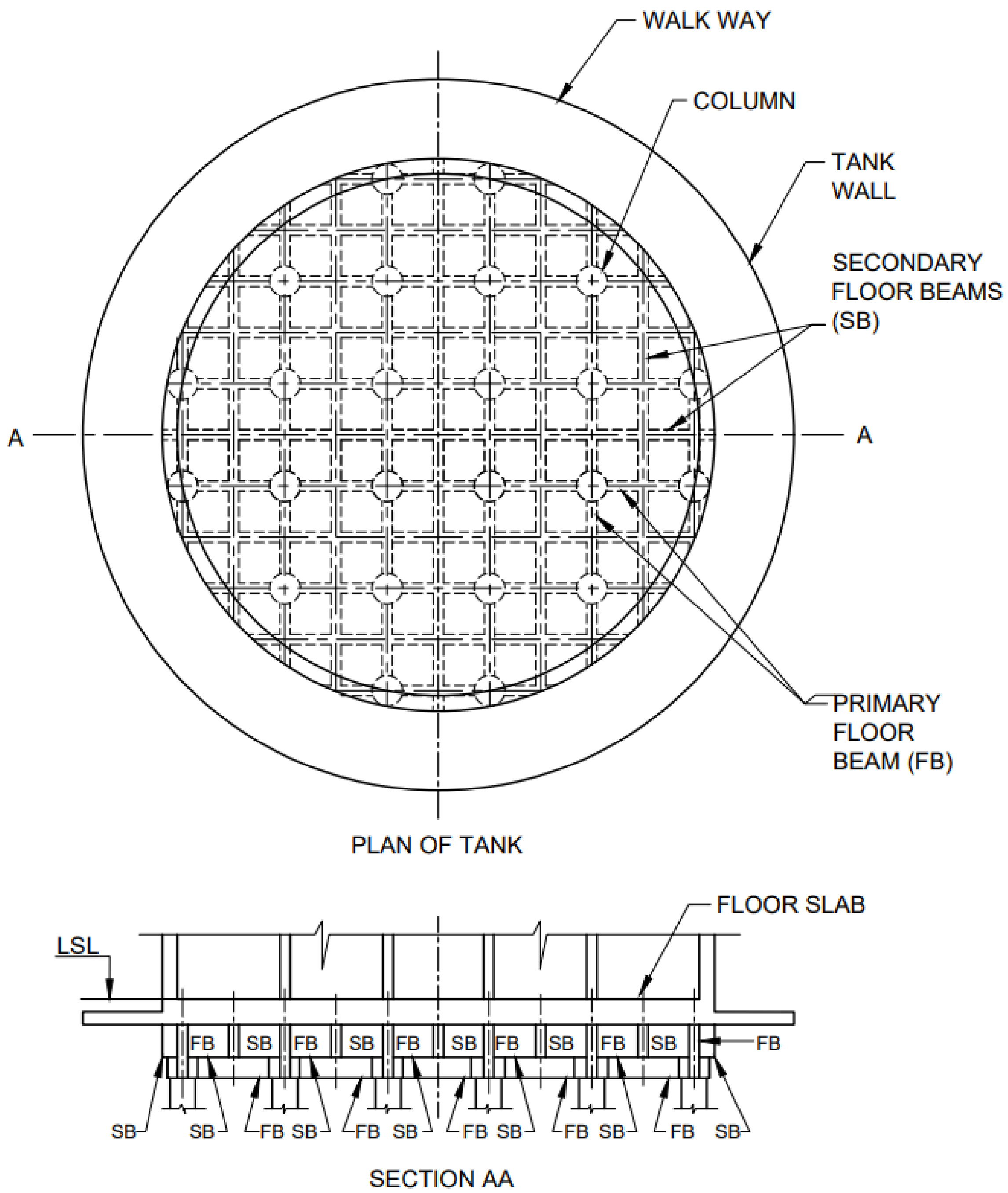


FIG. 5 FLOOR SLAB ON 24 COLUMN STAGING WITH SECONDARY BEAMS.

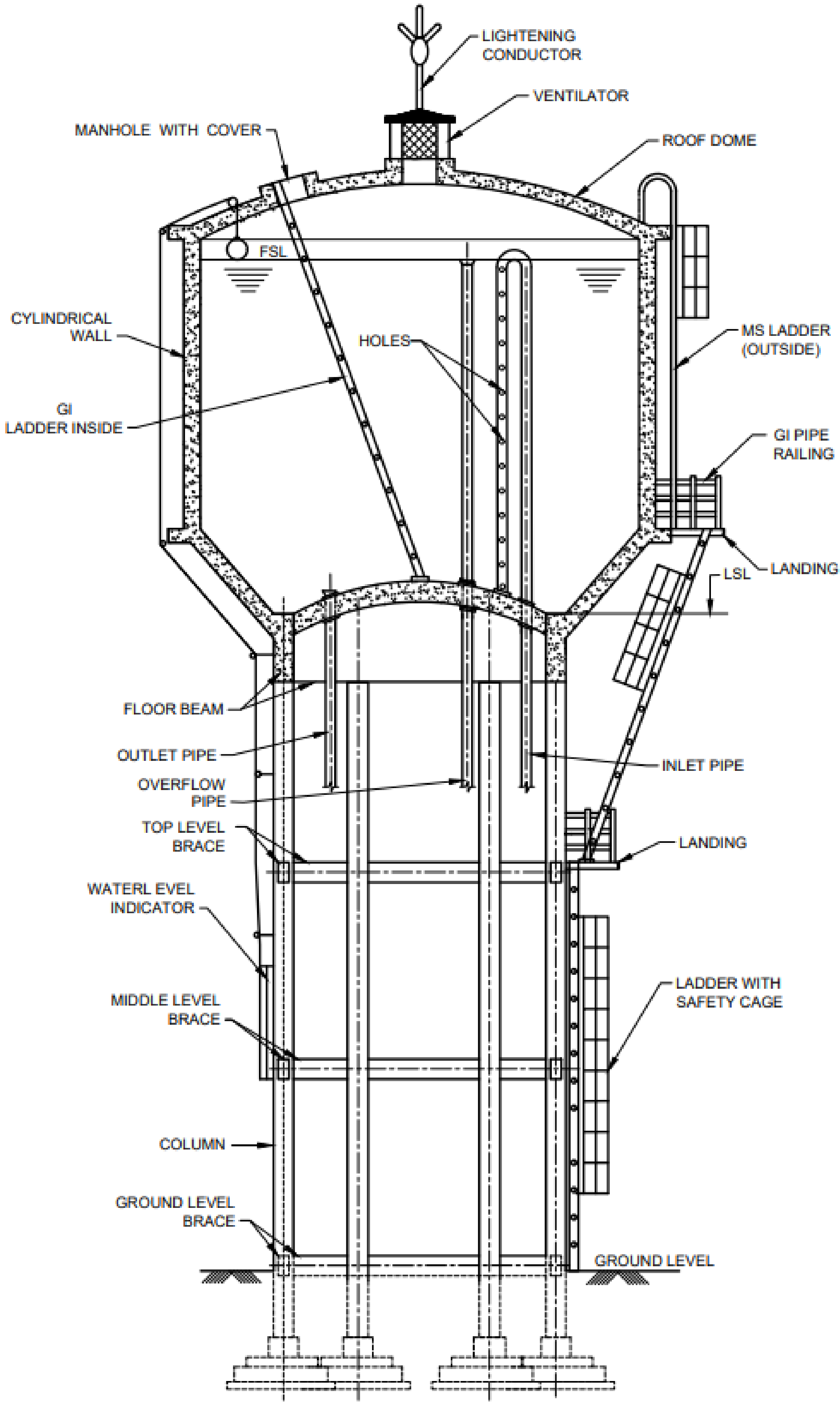
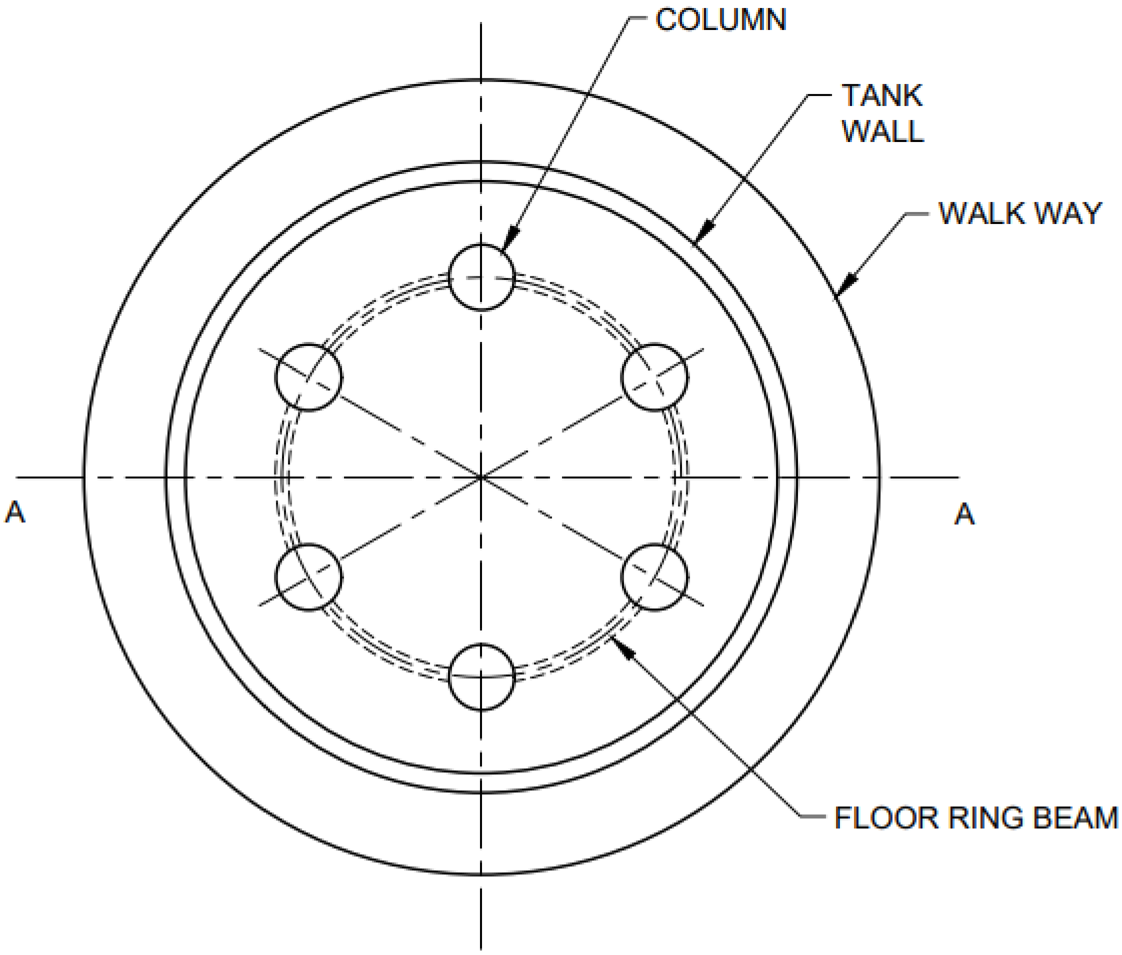
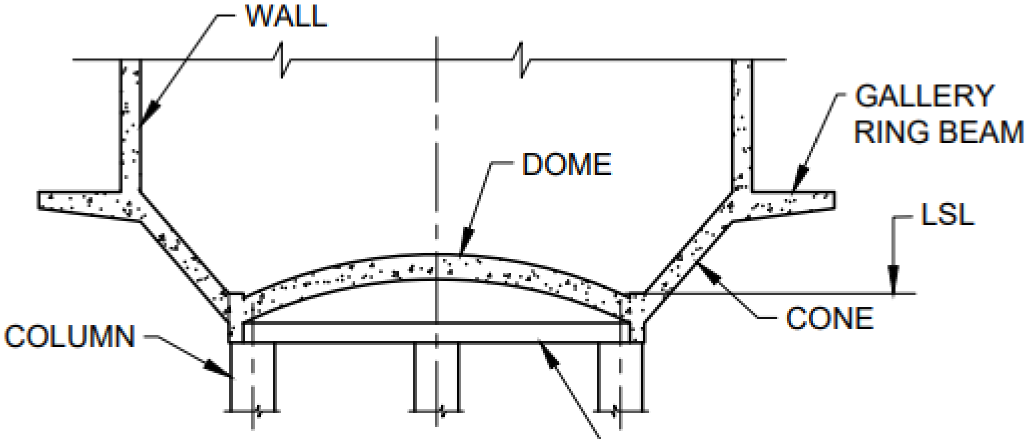


FIG. 6 TYPICAL SECTION OF INTZE TYPE WATER TANK.



PLAN OF TANK



SECTION AA

FIG. 7 INTZE TANK ON 6 COLUMN STAGING.

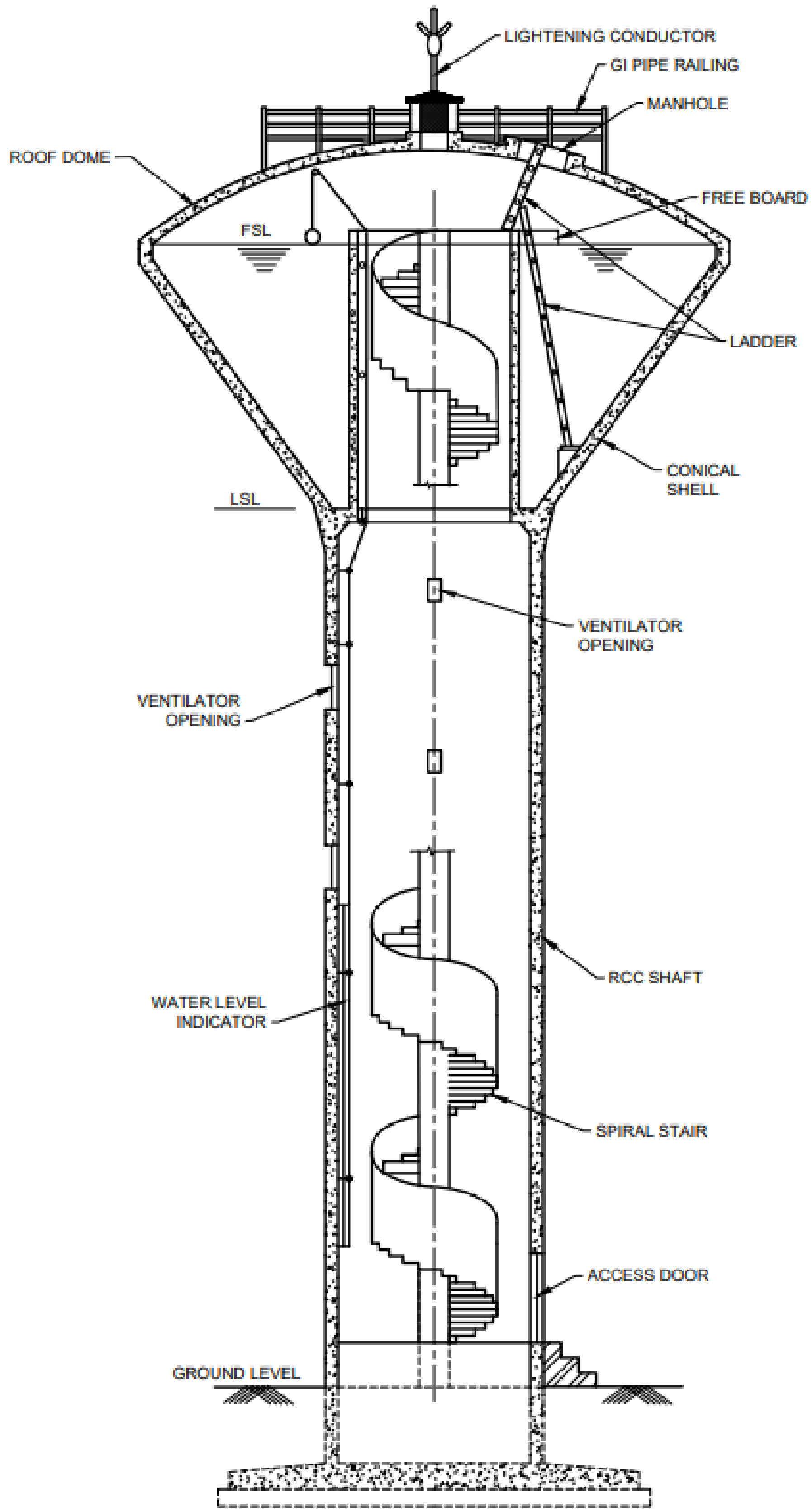


FIG. 8 TYPICAL SECTION OF CONICAL TANK ON RCC SHAFT.

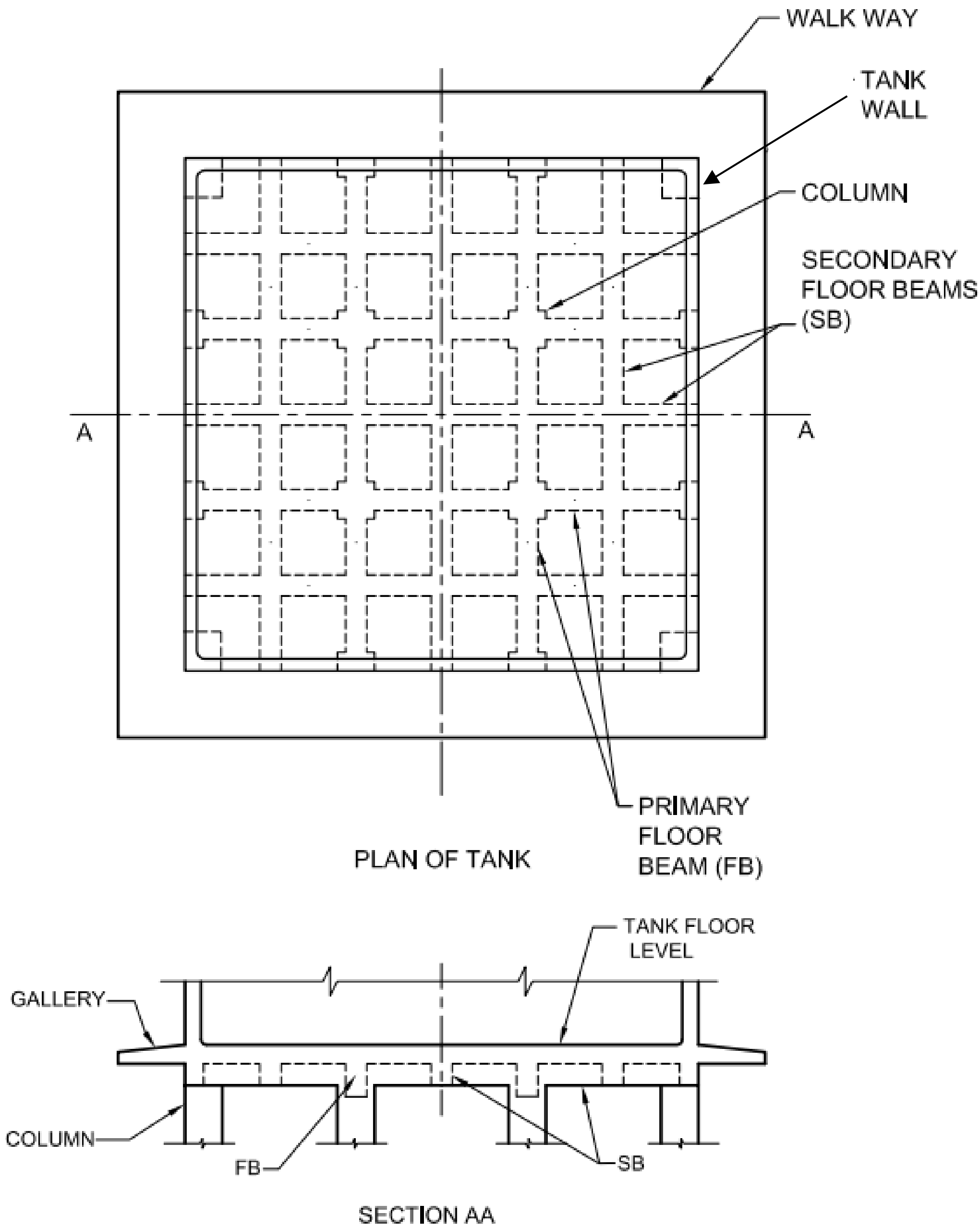


FIG. 9 SQUARE TANK ON 16 COLUMNS WITH SECONDARY BEAMS.

9 STABILITY OF STRUCTURE

Stability of the structure shall be checked as per provisions given in **9.1** to **9.6**.

9.1 Overturning

It shall be ensured as per IS 456.

9.1.1 For stability, the load combination is (1.2 or 0.9) DL + 1.0 FL+ 1.5WL.

NOTE – While considering earthquake effects, substitute EL for WL.

9.1.2 Overturning check will normally be critical with:

a) DL (no IL no FL) and wind load, and
b) DL + FL (no IL) and earthquake load.

Under the load combination for stability check, the maximum bearing pressure on soil shall not exceed the ultimate bearing capacity of foundation strata.

9.1.2.1 For overturning check, contribution of rock/soil anchor or anchor pile if accounted, their capacity to take tension (uplift) shall be reduced by 20 percent.

9.2 Sliding

Safety against sliding shall be ensured as per IS 456.

9.3 Probable Variations in Vertical Loads

To ensure stability at all times (as in **9.1** and **9.2**), the probable variations in dead load and liquid load during construction, service life, repair or other temporary occasions shall be accounted. Load factor for DL may be taken as 0.9 or 1.2, as may be more critical. Similarly, FL may be nil or in part or full with load factor 1, whichever gives more critical combination. Provisional dead load may be neglected, if DL helps in stabilizing. Wind and earthquake loading will be treated as overturning or de-stabilizing loads.

9.4 Lateral Load Resistance

In designing the frame for staging, provisions of adequate moment connections or a system of diagonal braces to effectively transmit all the horizontal forces to the foundations shall be made, without exceeding the limit of horizontal sway of each storey and total deflection at top. All junctions of columns and braces shall be designed and detailed so as to avoid large deformations, wide cracks, and failures within the junctions.

Connections or junction of members may be designed for force actions not less than that determined for any load combination. Direct tension and shear reinforcement in the junction shall be designed for higher force actions due to factored load combination.

9.5 Lateral Sway

For design wind and earthquake loads (each taken separately) the lateral sway at the top shall not exceed $H_{\mathrm{t}}/250$, where H_{t} is the total height tank up to top of container measured from top of the structural foundation. For earthquake force action, provisions in IS 1893 (Part 2) shall be applicable, and guidance given in this standard. For this sway limit, correction to stiffness due to cracking (as in**12.5**) shall be included, but $P - \Delta$ effect shall be excluded). For shaft type staging gross concrete section can be accounted for sway analysis, and natural period. The maximum storey drift limit shall be $H_{\mathrm{s}}/250$, where H_{s} is c/c storey height. (*see* also **14.1.3**).

For tanks on shaft, the deflection at top shall be limited to $H_{\mathrm{t}}/500$.

9.5.1 For checking the lateral sway at top of the elevated tank or storey drift the total wind force calculated on the structure (*see***10.4.1.7**) for strength design shall be applied with a load factor of unity. For earthquake base shear, refer IS 1893 Part2

9.5.2 For serviceability wind lateral sway at top of the elevated tank shall not be more than $H_{\mathrm{t}}/600$.

9.6 For unusual configuration of staging, the loss of stability of staging should adequately be studied.

10 LOADS

In structural design, account shall be taken of the dead load (DL), imposed load (IL also popularly known as live load), earthquake load (EL) or wind (WL). Liquid loads (FL or water load/pressure) do not fall in the classification either as DL or IL.

10.1 Dead Loads ($\boldsymbol{DL}$)

It shall be as per IS 3370 (Part 2). Loads due to possible rendering, finishes, lining in tank, plaster, piping, parapet, railing, stair, etc should also be considered. For concrete in contact with aqueous liquid, its wet density shall be considered. Wet density of reinforced concrete for members retaining aqueous liquids, should be determined. In absence of an appropriate value, wet density of reinforced concrete may be taken as 25.4 kN/m^3. Loads in this category are permanent and always present throughout the service life.

10.2 Liquid Load ($\boldsymbol{FL}$)

10.2.1 It shall be as per IS 3370 (Part 2). Density of water can be taken as 9.81 kN/m^3.

10.2.2 While liquid is overflowing, to match its rate of incoming, the heading of liquid above FSL can be 20 to 50 mm; such small rise can be neglected for load combinations. However, for the rare event of overflow blocked, the liquid level can rise to a level controlled by alternate path for overflow, and such a rise of level can be substantial. Freeboard of 150 mm to 300 mm may be considered normally. Higher free board may be specified for tanks having large horizontal size.

10.2.3 For deflection check, a serviceability limit state, DL is a permanent or long-term load. Also take 70 percent of FL as long-term. Other loads and remaining FL will be treated as short-term. FL will be taken 100 percent as long-term load, where tank may remain nearly full for long time as for some industrial tanks or that for firefighting.

For deflection of members due to long-term loads, effective modulus of elasticity for concrete should be obtained by short-term modulus of elasticity (E_c) divided by $(1 + \theta)$. Creep coefficient θ can be taken equal to 2.2. Due to horizontal forces, for estimating storey sway and horizontal deflection of staging, the creep correction shall not apply.

10.3 Imposed Loads (IL)

These shall be in accordance with IS 875 (Part 2). Snow loads shall be in accordance with IS 875 (Part 4).

10.3.1 Storage or piling up of material or sustained load over long periods, which may not be permanent, are called as storage imposed load (IL_s). Imposed load may also be due to processing, or provisional or operating equipment and its impact allowance (IL_p).

10.4 Wind load (WL)

It shall be estimated in accordance with IS 875 (Part 3). Load combinations shall take in to account both the tank empty and tank full conditions. The worst combination of the load on account of above shall be considered while working out the force action and the strength capacities or stresses.

10.4.1 Wind load shall be accounted as pseudo-static wind force as per **5** to **8** of IS 875 (Part 3). The elevated tank can be divided into different height zones. The wind force and its resultant are to be calculated for each of these zones component wise. Elevated tanks are treated as important structures for choosing k_1 factor.

10.4.1.1 For estimation of wind force on cylindrical wall, the roughness (ε) may be assumed as below depending up on excellent, good or average workmanship.

Table 1 Roughness of Outer Surface
(*Clause* 10.4.1.1)

Sl No.	Roughness	Description of the smoothness of outer surface
(1)	(2)	(3)
i)	2 mm	Super smooth form-finished surface without irregularities (for example, no dents in shuttering and of enough stiffness) such that surface profile is accurate. Formwork is designed for restricting deflections within 2 mm. The workmanship is excellent achieving smooth surface.
ii)	3 mm	Smooth surface finish without irregularities such that surface profile is accurate. Variations from specified surface are within

		3 mm measured over a curved length of 300 mm on surface. The workmanship is good.
iii)	5 mm	Nearly smooth surface finish. Variations from specified surface are within 5 mm measured over a length of 200 mm on surface. The workmanship is average.

Roughness coefficient (ε/d) should not be taken less than 0.000 2 for circular wall. For circular columns 0.004 may be assumed as minimum roughness coefficient (ε/d). Higher roughness may be accounted based on the expected workmanship and quality level of the work, or if specified by owner or decided by designer. All protrusions shall be grinded and surface rendered smooth.

10.4.1.2 Values of factors K_{d} and K_{c} shall be taken each as 1.0 for liquid storage elevated tanks. Factor K_{a} can be determined based on the area of container in elevation (including floor beam, wall and roof), as per Table 4 in IS 875 (Part 3). For all components of container and staging, same value of K_{a} can be applied. Combination factor (K_c) shall be 1.0.

10.4.1.3 While force coefficients (C_{f}) are estimated as per IS 875 (Part 3), for the members the effective valuesC_{f} shall not be less than the following.

a) Cylindrical wall –0.70,
b) Circular column – 0.50,
c) Square column – 1.70,
d) Braces – 1.20,
e) Rib of beams attached to slab – 1.2, and
f) Rectangular / square wall of tank – 1.2.

Reduction factor as per table 28 of IS 875 can be applied for cylindrical wall, and not for columns.

10.4.1.4 Wind load on ventilator, pipeline, stair, ladder, hoarding, neon signs, etc shall also be estimated for arriving at total wind load.

10.4.1.5 Wind load for limit state of serviceability ($W_{\mathrm{s}}L$), shall be taken for a return period of one year. For this the value of factor k_1 shall be as bellow.

	k_1 factor for basic wind speed m/s					
V_b	33	39	44	47	50	55
k_1	0.69	0.60	0.55	0.52	0.50	0.45

10.4.1.6 In load combination with wind load for serviceability, dynamic approach or gust factor need not be accounted, only pseudo-static wind load can be accounted.

10.4.1.7 For strength design (ultimate limit state) factor k_1 shall be taken for mean probable design life of 100 years as per Table 1 of IS 875 (Part 3) 2015. Load combinations shall be as per IS 3370 (Part 2).

10.4.2 Wind load can be estimated by gust factor as per **10** of IS 875 (Part 3), in case of conditions applicable in **10.4.3** and **10.4.3.1**. For dynamic wind loads (by gust factor) 'hourly mean wind speed' shall be used as per IS 875 (Part 3).

10.4.2.1 Wind load by gust factor method should be considered for ultimate limit state only. For the wind load to be considered, the staging shall be designed for tank full and also for tank empty conditions. Note that the natural frequencies for tank full and tank empty cases are different.
Gust factor method will be applicable to tanks staging having covered (clad) outer surface or shaft with total height of elevated tank more than $5 \times$ the outer diameter of shaft.
It is not applicable frame staging (columns-beams) an open structure and having staging height is less than $6 \times$ width.

10.4.2.2 For considerations of applicability of gust factor or wind dynamics, width of staging can be taken as centre-to-centre of outer most columns in plan along a wind direction; or for circular staging, the centre line diameter of the staging.

For applicability of gust factor criterion, by the limits of flexibility of the elevated tank for checking limiting value of height-to-width ratio, take the height of staging as the height to be accounted for open type staging, and total height (including container for tank on shaft. The container height has relatively no flexibility.

For further calculations of gust factor and wind force, total height of the structure including container shall be accounted.

10.4.3 For slender and flexible RC staging the wind load shall be estimated by gust factor method as per IS 875 (Part 3); except in cases given in (a) to (c) below.

a) Height of staging (open type) to width ratio is less than or equal to 6; or
b) Ratio of height of elevated tank (including container) to width of shaft is less than or equal to 5; or
c) For free vibration in first mode, natural period T is less than 1.0 sec; or
d) Optionally for cases as not required for gust factor method in **10.4.3.1**.

10.4.3.1 Where specially required or mutually agreed between the parties, or if decided by designer, design by gust factor can be done for any other case also.

10.4.3.2 For slender and flexible RC framed staging, the wind load estimated by dynamic response (gust factor method) as per IS 875 (Part 3) shall be accounted, for cases (a) to (f) as noted below:

a) Height of staging to width ratio > 6.0, and any value of natural period;
b) Height of staging to width ratio > 5 to 6, and natural period > 1.0 s;
c) Height of staging to width ratio > 4.5 to 5, and natural period > 1.5 s;
d) Height of staging to width ratio > 4 to 4.5, and natural period > 2.0 s;
e) Height of staging > 15 m, and natural period > 2.5 s;
f) Height of staging < 15 m and height of staging to width ratio ≤ 5 can be exempted from designing bydynamic wind response (gust factor method), if natural period < 1.6 s.

10.4.3.3 Natural period of vibrations shall be for first mode of free vibration under impulsive action as per IS 1893 (Part 2).

10.4.4 Staging shall have configuration and member sizes such that the first mode of natural vibrations shall not be the torsional mode.

10.5 Earthquake Load (EL)

Both tank empty and tank full conditions shall be considered as per IS 1893 (Part 2). Effect of convective mass of liquid should be considered for the design of staging. Both impulsive and convective effects shall be considered simultaneously as per the treatment referred to in **10.5.1** and **10.5.3**.

10.5.1 For earthquake analysis, the liquid mass shall be idealized by a two-mass model [*refer* to IS 1893 (Part 2)].

10.5.2 The earthquake load on the staging and its analysis shall be in accordance with IS 1893 (Part 1 and Part 2). For estimating the natural period of vibration of staging, its stiffness should be reduced on account of cracking (that is, deflections to be enhanced). Correction in deflections due to $P-\Delta$ shall not be applied, forestimating the natural period. However, stiffness of floor beams and members of container shall not be reduced.

10.5.3 Earthquake base shear on staging shall be estimated for the following load combination:

$$1.0\,DL + 1.0FL + \ 1.0IL_{\mathrm{s}} \ + \ 0.7\,IL_{\mathrm{p}}$$

This base shear shall be multiplied by an appropriate load factor for a load combination. In estimate of base shear, the fluid load should further be limited to a quantity (factor 0 to 1) as may be taken in a particular load combination with earthquake load.

10.5.3.1 If imposed loads are other than live loads on roof, and of nature like a process or operations or equipment (IL_{p}), an appropriate part (may be 0.7) of such IL_{p} excluding impact allowance shall be accounted for while estimating base shear in **10.5.3**.

10.5.3.2 Though vertical pipelines are non-structural components of elevated tanks, there performance is also influenced by the requirements for a minimum lateral stiffness (drift limits) for structural systems and requirements that pipelines accommodate the anticipated structural drift; the stiffness requirement is more restrictive for higher risk categories.

10.6 Blast Load or Vibration Effect Forces

10.6.1 Where specified, design shall be checked for force actions caused due to vibration and impact excitations by blast action (*see* IS 6922) as experienced near mines, collieries and in the close proximity of railway tracks, etc or explosion (*see* IS 4991). This type of load shall be assumed to act in place of earthquake or wind, if it gives critical action in any member of the structure.

NOTES

1 In most cases, the effect of vibration or blast due to the charge normally permitted per delay, may be significantly less than the earthquake consideration.
2 The structure will be designed for the explosion only if required under a contract as specification of owner (or as decided by a designer,) by specifying the probable charge and its distance.
3 For tanks located near mines, in addition to vibration forces, effect of mining subsidence could also be given due consideration, if the necessary data from experts is made available by owner to the designer. Also *refer* to IS 1904.

10.6.2 If specified by the owner, for design against explosion, the survival of staging shall be checked for condition of loss of one column or a significant portion of the shaft staging. This design condition will require substantial increase in the cost of staging. In load combination for such a condition, the load factor for any load shall not be more than 1 and imposed load (IL) can be neglected. The partial material factor can also be reduced to 1 for steel and 1.2 for concrete.

10.6.3 The design for blast or explosion shall be done, if mutually agreed between the relevant parties, or as mentioned in the contract.

10.6.4 If to be designed for blast or explosion, elevated tanks in earthquake zone II shall also be designed for all the requirements for earthquake zone III.

10.7 Construction Loads

Temporary loads resulting from construction activity shall be considered in design of structural components.

10.8 Temperature Shrinkage (TS)

The effects due to temperature, shrinkage, creep, etc, where critical may be accounted for in the load combinations for serviceability limit state. In RC structures, significant part of actions induced due to temperature and shrinkage are reduced due to creep and cracking. Here design value of TS is to be accounted, which is relaxed by multiplying by 0.20. For temperature and shrinkage, following load combination can be considered for serviceability limit state.

$$1.0\,DL + 0.7\,FL + 0.3\,IL + 1.0\,TS$$

10.9 If applicable at a particular location, dust load should be accounted on roofs.

11 LOAD COMBINATIONS

Load combinations and partial safety factors shall be as per IS 3370 (Part 2). For load combination with wind or earthquake, the columns and braces shall be checked by limit state design method with effect of sway (or $P - \Delta$ effect) as applicable (*see* **12.2**).

12 ANALYSIS

12.1 General

12.1.1 Force actions (bending moments, torsion, shear forces, direct forces) in the components of structure shall be adequately analysed. In particular, adequate consideration shall be given to the effects of monolithic construction (rigidity at junctions) in assessment of member forces.

Where movement joints are given in or between members, these joints shall be designed to withstand movements during life, without inflicting damage or loss of durability. Joints shall be adequately sealed such that intended relative movements are permitted, avoiding loss of durability, and loss of stored liquid.

For analysis of staging, some guidelines on structural modelling are given in **14.1**.

12.1.2 The designer should correctly estimate the loads and static equilibrium of structure particularly in regard to overturning of overhanging members. The design shall be based on the worst possible combination of force actions, arising from vertical and horizontal loads acting in any direction when the tank is full as well as in empty case.

12.2 For the analysis of frame, including $P - \Delta$ effect, modulus of elasticity of concrete will be taken as per IS 456. No correction for creep is necessary.

12.2.1 For accounting $P - \Delta$ effect in framed staging, the moments in columns can be calculated as per the provision in **39.7** of IS 456, if conditions in **14.2.1** are fulfilled. Alternately, detailed $P - \Delta$ analysis may be done.

12.3 For earthquake design, inertial eccentricity is distance between centre of mass and centre of rigidity (or stiffness) measured in a horizontal plane. For tank and staging symmetrical about each of two principal axes in plan, the inertial eccentricity will be negligible and can be neglected in design. If the structure has an eccentricity, same shall be accounted without magnification, in the dynamic analysis of staging. The effect of vertical pipe assemblies on eccentricity can be neglected.

12.4 For staging in plan, columns and braces shall be arranged such that there is symmetry in layout and stiffness considered about each of the two principal directions. If this is not satisfied, the structure shall be treated as irregular (*refer* to IS 1893 Part 1). Multiple modes (3 to 5) of vibration shall be considered for dynamic analysis. The design will be very much involved and need much higher level of expertise in analysis of such structure, which is not dealt in this document. The first mode simplified analysis is for a regular structure only.

12.4.1 Irregularities in plan and vertical irregularities in the structure shall be avoided, unless detailed dynamic analysis is performed to account them [*refer* IS 1893 (Parts

1 and 2)].

12.5 For staging consisting of columns and braces in RC, for analysis the stiffness of members shall be reduced on account of cracking. Member stiffness can be worked out on the basis of second moment of area (popularly called as moment of inertia) of the gross concrete area of the member cross-section, which is reduced on account of cracking. For stiffness purpose, the second moment of area of column can be reduced to 0.9 times, and for braces to 0.6 times.

12.5.1 For shaft type staging, stiffness shall not be reduced on account of cracking.

13 BASIS OF DESIGN

13.1 Staging and other reinforced concrete members including foundation shall be designed by limit state method in accordance with the requirements of IS 456.

13.2 For members of foundation, with the required load combinations under limit state of serviceability, the stresses in steel shall not be more than $0.55f_{\mathrm{y}}$. No specific check on crack width is required for medium exposure condition. In case of severe environment, the crack width in foundation members shall be limited to 0.2 mm.

13.3 If designed for severe exposure conditions, in members of staging, the steel stress shall not be more than $0.55f_{\mathrm{y}}$ under serviceability limit state.

13.4 Elevated tanks are highly sensitive to differential settlement. Hence, a maximum of 50 mm settlement shall be the limiting criteria for arriving at the allowable bearing pressure.

13.5 Construction Joints

Construction joints in members should be as less as possible. Their positions should be specified in design and drawing. Designer shall take into account the possibility of lower strength in direct shear, tension and higher crack width at such joints.

13.6 Shear Strength

For a member (beam/brace/wall) subject to bending in combination with direct tension, shear strength of concrete shall be reduced as per IS 3370 (Part 2).

13.6.1 For member subject to direct tension, the critical section for shear force should be taken at face of support and not at a distance from face of support. If shear force exceeds the shear resistance of concrete, stirrups are designed to take shear and spacing of such stirrups shall not be more than half the effective depth of member, but not less than 75 mm for small depth member. Stirrups shall also conform to the requirements of IS 13920.

13.6.2 At the location of construction joint (with interface roughened) in a member (beam/brace/slab, etc), the shear strength of concrete (excluding contribution of shear reinforcement) shall be assumed two-third only.

14 FRAMED STAGING

Framed staging consists of column and braces. Frame coupled with shear wall can also be provided. In case of dual system, horizontal shear shared by frames will be determined by relative stiffness of frames and shear walls. However, the frames (under dual systems) shall be designed for a minimum horizontal shear not less than 1 percent (as A_{h}) of the vertical/gravity load on all columns of staging together. However coupled shear wall shall be designed for a base shear resisted by it in proportion to the total stiffness of the staging.

14.1 Structural Modelling

The force actions in the members get affected by the simplifications made in the modelling. Simplifications known as acceptable practice, can only be made as given below:

a) Columns at top of staging (at junction with container member) may be assumed rotationally fixed, that is, stiffness of member connecting column tops is taken extremely high. Such an assumption reduces the design moments in the braces at a level just below container (for example, top most brace level). In such case, top brace should be designed for higher moments (by 33 percent) or it may be same as the brace at a level below it, if such an assumption is made.
b) If stiffness of container members is underestimated (say floor beams considered as rectangular sections, neglecting stiffness contribution of supported slab or dome), the column moments at the top junction will be under estimated and should be enhanced by 25 percent for design, and the moment in brace below will be little over estimated.
c) Normally it is permissible to assume the base of column fixed at the level of mid height of structural foundation (footing /pile cap /foundation beam, etc), for the analysis of staging. Such simplification, underestimates the moments in the brace just above foundations or at GL. Hence, design moment in first level brace (near to GL or nearly plinth level), should be suitably enhanced. (*see* **14.4.6**)
d) Where eccentricity between centre of mass and centre of rigidity is negligible (as per **14.1.2** and **14.1.2.1**), the stiffness of column of stair (if not part of main staging) can be neglected in analysis of main staging.

14.1.1 *Analysis and Design*

For analysis and design, the frame along the centre line of members shall be considered. The length of a member shall be the distance between the centre-lines of the members connected at the ends.

a) The junctions of columns and braces have finite size. A joint can be assumed to be rigid or the rigidity factor for junction can be reduced to a value between 0.7 and 1.0. In most cases the width of brace shall be smaller than width of column, and in such cases brace can be designed for a critical section (within junction) at 50 mm distance inside from the face of junction.

b) For analysis of staging, the height of first lift of column shall be taken as distance between centre of structural foundation to the centre of next brace (at or within ground).
c) Design column moments shall be at a section 100 mm distance inside from top face of column pedestal above foundation. Near staging top, design column moments shall be at a section 100 mm distance inside from face of container member (like floor beam or wall). At the column brace junction the critical section for column design shall be at the middle of the junction.
d) For floor beams of tank, if the span-to-depth (c/c) ratio is less than 2.5, it requires reduction of lever-arm for calculations of tensile steel.

14.1.2 Stair may consist of spiral type on an independent column or other type on more independent columns (excluding columns of staging). If such columns are not placed symmetrical about two principal axis in plan of staging, such arrangements induce eccentricity between centre of mass and centre of rigidity or stiffness (as in **12.4**) for the consideration of dynamic analysis.

14.1.2.1 Configuration of staging shall be symmetrical along two mutually perpendicular directions to avoid eccentricity in the behavior. Effect of such eccentricity can be significant if columns carrying load of stair are connected to the staging of tank and not located at centre of the staging. In such cases eccentricity of mass and stiffness shall be accounted for in the analysis of staging.

Effect to cause eccentricity can be small for spiral-stair on a single column even if not at centre of staging. Where configuration of columns in staging is symmetrical along two mutually perpendicular directions, such eccentricity may be neglected, if staging has total cross-sectional area of columns more than 8 times the cross-sectional area of columns of stair.

14.1.3 The horizontal deflection (sway) or storey drift (with reduced stiffness as per **12.5**) will be calculated for the limit state of serviceability, for load factors unity.

For the design horizontal wind or earthquake loads, the drift in a storey (computed elastically based on first order analysis that is, without $P-\Delta$ effect) shall be checked for limits as specified below:

a) If the drift is not more than 0.25 percent of storey height (that is, height/400) and **14.2.1** is satisfied, detailed $P-\Delta$ analysis need not be done, and additional moments may be estimated as per Clause **39.7.1** of IS 456.
b) If the drift is between 0.25 percent and 0.33 percent of storey height (that is, height/300), or if any one requirement in **14.2.1** is not satisfied, detailed $P-\Delta$ analysis is needed.
c) If the drift is more than 0.33 percent of storey height, detailed $P-\Delta$ analysis shall be done with Δ enhanced by 1.20 times.

14.2 $P-\Delta$ Effect

Staging of columns and braces shall be designed for $P-\Delta$ effect. In this standard wherever detailed $P-\Delta$ analysis is specified, it means a second order analysis accounting the effects of horizontal deflection. The simplified calculation of additional moments (as in **39.7.1** of IS 456) does not constitute a 'detailed $P-\Delta$ analysis', and it may be in lieu of detailed analysis.

Where conditions specified in **14.2.1** are satisfied, $P-\Delta$ effect is likely to be small, and the additional moments as per **14.2.1** should be accounted in design, in lieu of detailed $P-\Delta$ analysis.

14.2.1 For design of framed staging, the requirement of $P-\Delta$ analysis can be deemed to be satisfied by estimation of additional moment applied at each column beam junction. This shall be maximum of that estimated in **14.2.1.1** and **14.2.1.2**, and all of the following conditions are satisfied. If any one of following conditions is not satisfied, detailed $P-\Delta$ analysis shall be carried out.

a) The elastically computed first order lateral deflection of each storey is not more than $1/400$ times (that is, 0.25 percent of) the storey height.

b) The stability index 'Q' for all the middle storeys (each taken individually) is less than 0.10; (*see* **E-2** of IS 456).

$$Q = \Sigma P/[(H/\Delta)\, h_{\rm s}]$$

where

ΣP = sum of axial loads on all columns in the storey,
H/Δ = total lateral force within the storey (that is, storey shear) per unit of elastically computed first order lateral deflection, and
$h_{\rm s}$ = height of the storey ($\rm c/c$ of braces).

c) Clear height of column of staging between braces should not be more than ten times the smaller size of column cross-section. This requirement may be relaxed for few columns (less than 50 percent of the number of columns in the staging) if the size as average of all columns satisfy the requirement.

d) Brace size shall be such that any one of the following is satisfied:

1) The percentage of concrete in braces should not be less than 40 percent of total concrete in columns and braces, and
2) Depth of middle braces shall be not less than 0.85 times the column size; and depth of any brace shall not be less than 0.65 times the column size; and Middle braces are other than those just above foundation to GL, and the top braces which are just below the container.

14.2.1.1 Additional moment at each column junction estimated as per **39.7.1** of IS 456, shall not be less than as specified in 14.2.1.2.

14.2.1.2 Additional moment at a column junction shall not be less than moment due to product of additional 20 mm eccentricity and load on column. This accounts for the construction errors along with $P-\Delta$ effect [*refer* IS 1893 (Part 2)].

14.2.2 Additional moments in columns at junctions are estimated as per **14.2.1**. These additional moments at the ends of columns will induce additional moments in braces also, which should be accounted for in the brace design. Hence, these additional moments shall be applied at each column junction while analysing the frame for horizontal loads.

14.2.3 Whenever required, detailed $P-\Delta$ analysis shall be done for each load combination as input load. Separate $P-\Delta$ analysis for a horizontal load and superimposing the force action (result of analysis) for a combination is not valid.

14.3 COLUMNS

Columns of circular sections are preferable, or square sections may be used. For rectangular sections the aspect ratio (smaller dimension to larger dimension) of the cross section shall not be less than 0.5. These shall be oriented such that the total stiffness of staging to horizontal force in two principal directions shall be nearly equal.

14.3.1 *Forces and moments in columns*

The entire load on tanks shall be considered to be transferred to the columns in the manner in which the floor of the tank contributes to each column. The effects of continuity of the beams and wall at the top of the columns, if any shall be accounted in calculating the reactions on columns. For continuity effect, proper stiffness of members meeting at junctions shall be accounted. In addition to tank load, force actions due to wind, earthquake, or vibrations shall be considered**.**

14.3.2 Columns shall be designed for a minimum moment, estimated for eccentricity equal to $h_{sc}/400 + d/30$ or 30 mm, whichever is lower. Where, h_{sc} is unsupported length (clear height between braces) of column; and d is lateral dimension or diameter or size of column.

It is sufficient to ensure that moment equals or exceeds the minimum, about one axis at a time. For design, bending moment shall not be less than the product of most critical (maximum) axial load and the minimum eccentricity specified here. In limit state design, the load will be the maximum factored axial load.

14.3.3 *Horizontal Loads*

Forces and moments resulting from horizontal loads may be computed for the critical direction and used in the design of the structure. Analysis may be done by any of the accepted methods (like moment distribution, stiffness matrix, etc), considering the staging as space frame.

14.3.3.1 Horizontal loads shall act on all parts of the tank as well as the staging. Horizontal loads on pipelines, stair, ladder etc shall also be estimated to be the part of total horizontal force.

14.3.3.2 Additional axial forces in columns, due to horizontal loads can be calculated by equating the moments due to all horizontal forces above the level of considerations, to the restraining moment offered by axial forces in columns, unless frame is analysed as space frame in which case effect is already accounted for in the analysis.

14.3.3.3 Due to horizontal load, bending moment in a column shall normally be critical (maximum), if in plan the column lies on the bending axis of staging as a whole, or the column is nearest to the bending axis. This criterion will govern the direction of horizontal force (perpendicular to the bending axis), with respect to the position of the column for analysis.

14.3.3.4 Due to horizontal load, additional axial load in a column shall be maximum, if in plan the column is at maximum distance from the bending axis of staging as a whole.

NOTE – This provision does not apply to staging on square grid having braces in alternate bays, and not throughout (*see* **A-2.2.1.1**).

14.3.3.5 All columns of a staging shall have nearly the same foundation level. Length of all columns centre-to-centre from foundation to GL braces shall be approximately equal.

If variation in the column lift is more due to variation in depth of foundation for different columns, following can be done. An option can be to fill lean plain concrete in foundation pit, which is deeper, such that subsequently the column lifts will be nearly equal. In absence of this option, it should be ensured that from geotechnical considerations the difference in foundation levels is permitted.

Further, the design should be based on the analysis of staging for horizontal loads (wind and earthquake) accounting the actual length of each column (in bottom storey) for lateral sway, and modal analysis of staging vibration shall be carried out. In such a case, the column of least height would resist much more shear force and bending moment; as well some GL braces shall carry significant direct tension.

14.3.4 Design of columns shall be governed by following guidelines:

a) For column size less than 500 mm, the strength capacity of column shall be reduced by multiplying a coefficient (C_r):

$$C_r = \sqrt{(d/500)}$$

where d is column size (diameter or smaller size of section) in mm.

Column size less than 400 mm is not preferable. (C_r shall not be more than unity). Columns size less than 400 mm (≥ 300 mm) may be provided for tanks up to 10 m³ only. For achieving above, while checking the column section by interaction diagram, the M_u and P_u should be enhanced by dividing them with C_r.

b) The columns inside the container and connected at floor and roof such that all the horizontal forces (> 99 percent) are resisted by the walls of container, are non-sway columns. For such columns, reduction in strength capacity:

$$C_r = \sqrt{d/300}$$

where

d is column size (diameter or smaller size of section) in mm. The size of such columns shall not be less than 200 mm.

14.4 Braces (Tie Beam)

Each column shall be connected by minimum two braces, which shall be in two separate vertical planes. As far as possible these braces shall make an angle 60° to 120° between them, preferably 90° in the plan.

14.4.1 Staging height above foundation (or GL brace) to container bottom, if greater than 12 times the column size, the column shall be rigidly connected by horizontal braces (tie beams) suitably spaced at intermediate levels.

14.4.1.1 If all columns are on a circle, and the angle between the braces exceeds 135° (that is more than 8 columns on a circle), the response reduction factor shall be reduced. If the angle exceeds 150° (that is more than 12 columns), detailed $P - \Delta$ analysis will be necessary (even if storey sway is small). Also, for earthquake design modal analysis shall be required, accounting distortion of staging shape in plan. For such a case (angle more than 150°), torsion in brace should not be neglected (torsional stiffness not released at ends) for brace design. As the said angle increases, the demand on torsional stiffness and capacity of brace also increases.

To reduce the angle between braces, in addition to circumferential braces additional braces may be provided, which are called internal braces [*see* Fig. 30(e) & 30(f)]. With provision of internal braces, the conditions of detailed $P - \Delta$ analysis and brace width as in **14.4.3** (iv) shall not apply.

14.4.2 Force actions (bending moments, shear force etc.) in a brace due to horizontal loads shall be critical while horizontal force on staging is acting in a direction parallel to the vertical plane of the brace. The moments in braces shall be the sum of moments in the upper and lower columns at the junction resolved in the direction of horizontal braces.

14.4.3 Analysis and design of braces shall be governed by following guidelines.

a) Brace width shall be minimum 200 mm or more, as required for constructability. For better constructability it is advisable to have 80 mm horizontal gap between longitudinal bars to facilitate concrete pouring and vibration by immersion vibrator. For convenience in construction, all braces in a staging may have a standardized width. For tanks larger than 250 m^3, the width of brace should preferably be 230 mm or more.
b) Width of un-flanged braces shall be not less than $1/25$ of the clear distance between junctions or other crossing brace. For brace with a flange, the width of web of braces shall be not less than $1/36$ of the clear distance. For brace having flanges on both faces (top and bottom) width restriction (as a ratio of length) shall not apply.
c) For rectangular section of brace (having no flanged) the width-to-depth ratio shall not be less than 0.30. However, for economy, this ratio should not be

much higher.

d) For staging having more than 8 columns on a circle (for Intze tank or cylindrical tank), the width of brace shall not be less than 250 mm, and minimum 300 mm for more than 12 columns, if other than circumferential (that is, internal) braces are not provided.

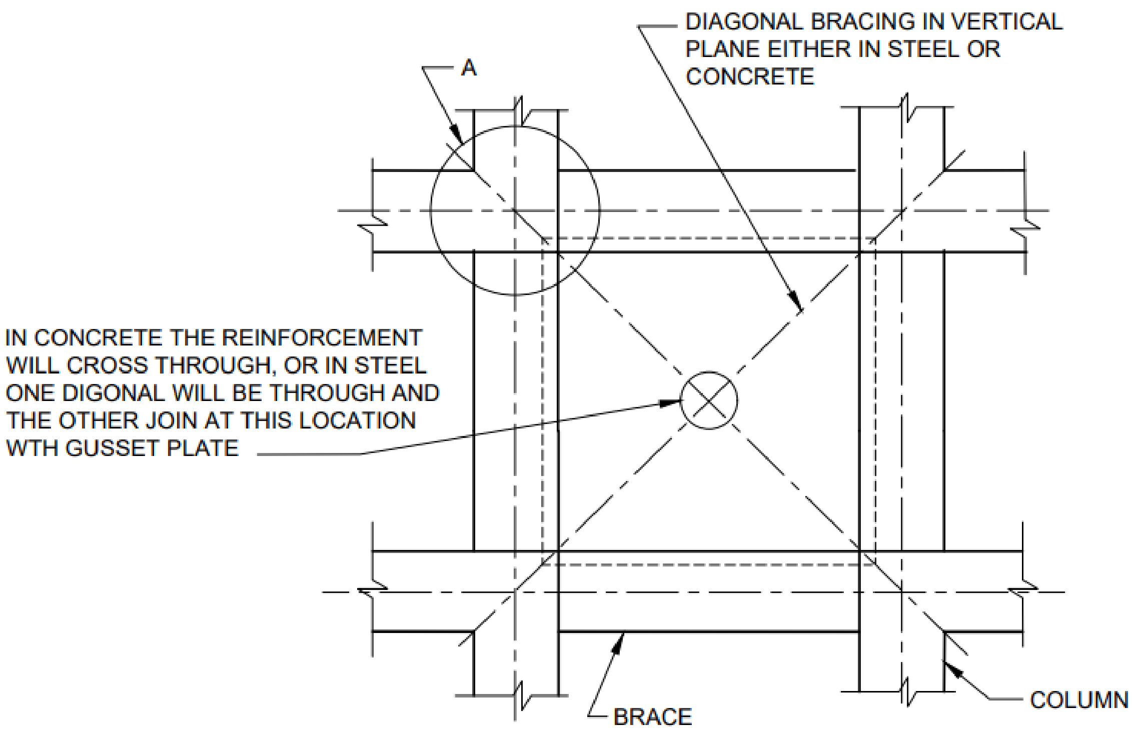
14.4.4 Moment and shear arising from self-weight and local vertical loading (if any), shall also be accounted for in the design.

14.4.5 In the set of ground braces or braces just above foundation, each shall be designed for a minimum direct tension equal to one fifth of base shear in the column to which it connects. Such tension will be in addition to the design force actions (including moments) on the brace. The design tension shall be higher if the storey heights above foundation to the brace are differing for columns.

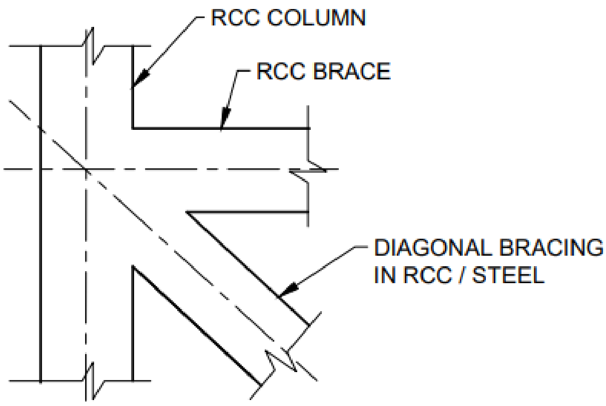
14.4.6 Enhancement of design moments in ground brace shall be done, if individual footings are provided to each column, and for moment analysis column is assumed to be fixed (that is, foundation interaction for rotation is not accounted.)

NOTE – The enhancement of bending moment in brace can be worked out by applying additional moment equal to $K_{\mathrm{b}} \times M_{\mathrm{b}}$ at the junction of GL brace with column and distributing the moments at the junction. Here, M_{b} is the bending moment at the column base (junction with foundation) due to horizontal action. Value of K_{b} will decrease with the stiffness of foundation soil and is a function of soil-structure interaction, and in absence detailed analysis it may be assumed as 0.20.

14.4.7 For staging in earthquake zones IV and above or where basic wind speed is 50 m/s or more, twin diagonal bracings (in vertical plane) of steel or RC may be provided, in addition to the horizontal bracing. Typical sketches of diagonal vertical bracing are shown in Fig. 10 (A) and 10 (B). *(IS 1893-3 is referring to IS 11682 for this item.)*

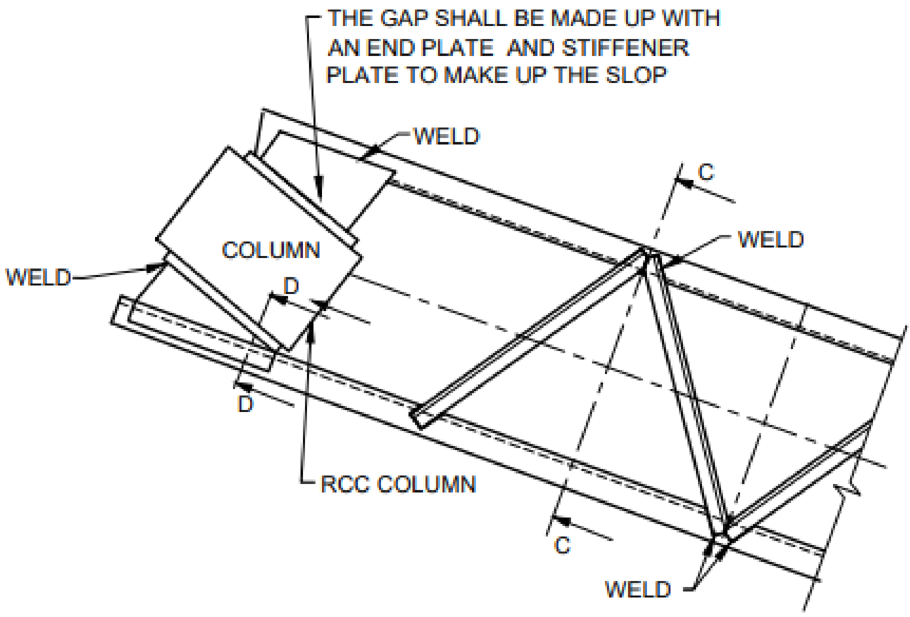


ELEVATION OF A PANEL OF STAGING
(ONLY FOR ALTERNATE BAYS IN PLAN)

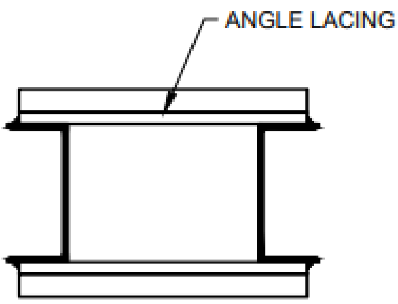


DETAIL AT A

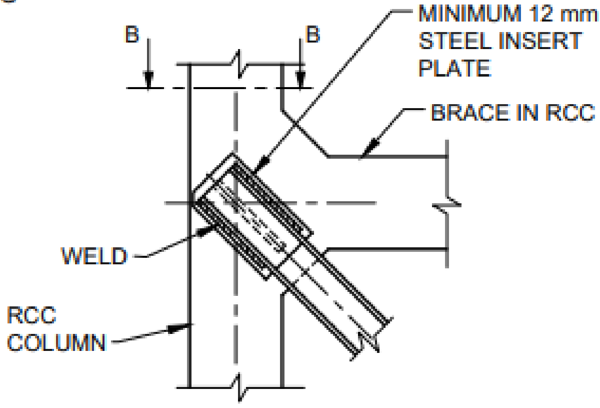
10(a) Typical Details of Diagonal Bracing in Concrete



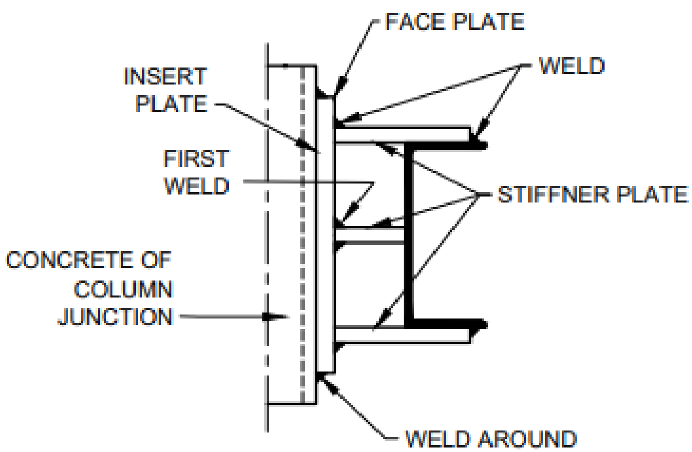
VIEW BB (IN PLANE)



SECTION CC



CONNECTION OF DIAGONAL
BRACING IN STEEL TO COLUMN



SECTION AT D D

10(b) Typical Details of Diagonal Bracing in Steel

FIG. 10 TYPICAL DETAILS OF DIAGONAL BRACING IN CONCRETE OR STEEL.

14.4.8 Stirrups shall be conforming to the requirements of IS 13920.

14.5 FOUNDATION

Foundations shall comply with the requirements of IS 1904. For staging with columns on a circle, requirements of towers and silos shall be complied with. For framed staging, requirements of RC framed structure shall also be complied with

Enough safety of all types of foundations shall be ensured even in the condition of heavy liquid leakages adversely affecting the safe bearing pressure, settlement etc.

14.5.1 Individual footings may be provided for columns, and shall be designed for critical combinations of load and bending moments as per requirements of IS 456.

As far as possible the footings with sloping top surface (that is, footing section trapezoidal) shall be avoided, as the concrete of sloped footing cannot be vibrated properly. Block or stepped footings are preferred. Though square footings are common, rectangular or trapezoidal shape in plan can be provided.

Combined footing with or without tie beam, or strip foundation may be provided where required. Mat foundation or raft foundation in accordance with IS 2950 may be provided. Ring foundations may comply with IS 11089. Design actions (forces) can be obtained from equations given in other acceptable documents or by the method of finite element analysis.

14.5.2 All columns shall be tied together above foundation level and near ground by structural members such as continuous horizontal braces. As far as possible such a set of braces shall be partly or fully within ground level except if brace is just at top of foundation, if foundation depth is small. Clear height between foundation top and such a brace shall not be more than 3 times the size of pedestal or column as applicable between the foundation and the brace. If continuous strip (or annular strip) or mat or raft foundations is provided, the requirement of continuous tie to columns may deemed to be satisfied, and restriction on clear height of column (3 times the size) up to next brace shall not apply. However, continuous ground braces shall be provided in all cases (*see* also **8.4** and Fig. 11).

14.5.3 Foundation shall be proportioned such that, under vertical loads of elevated tank (with tank full as well as empty) at serviceability limit state, the maximum pressure on the soil is within the net allowable bearing pressure of founding strata. While effects of horizontal forces (un-factored) are accounted for, the maximum pressure on the soil shall be within the net safe bearing pressure (not necessarily net allowable bearing pressure). Under stability check (with factored moments), or load action in ultimate limit state combination, the maximum pressure on the soil shall be within the $0.9 \times$ ultimate bearing capacity of founding strata (*see* Fig. 12).

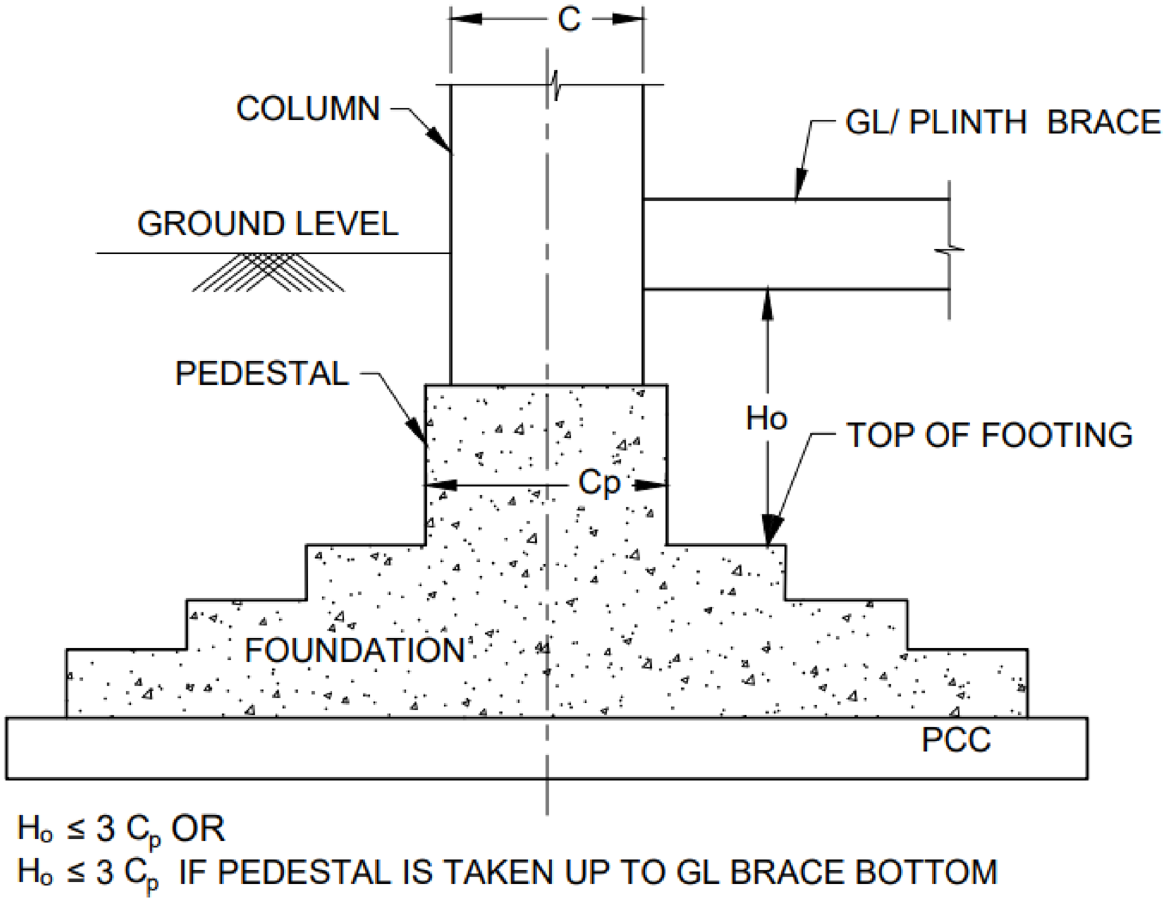


FIG. 11 PEDESTAL AND GL BRACE.

14.5.3.1 From tests gross ultimate bearing capacity can be arrived at. Safe bearing pressure will be obtained by applying a factor of safety 2 to 3. Factor of safety may be higher than 2 for individual footings and will also depend upon method of testing and uniformity of strata. Allowable bearing pressure shall be arrived at from permissible settlement considerations, but it shall not be more than safe bearing pressure. At the founding depth, net pressure indicate the capacity in addition to the existing burden of soil, that is, weight of overburden of existing soil at founding level due to soil height from founding level to GL.

14.5.3.2 Allowable bearing pressure (consideration of settlements taken) shall be considered in serviceability state, or where mostly gravity loads are considered, and maximum design horizontal loads like wind and earthquake are not considered.

When maximum wind or earthquake is also considered (un-factored actions), the safe bearing pressure with permissible increase in bearing pressure, shall be accounted in design.

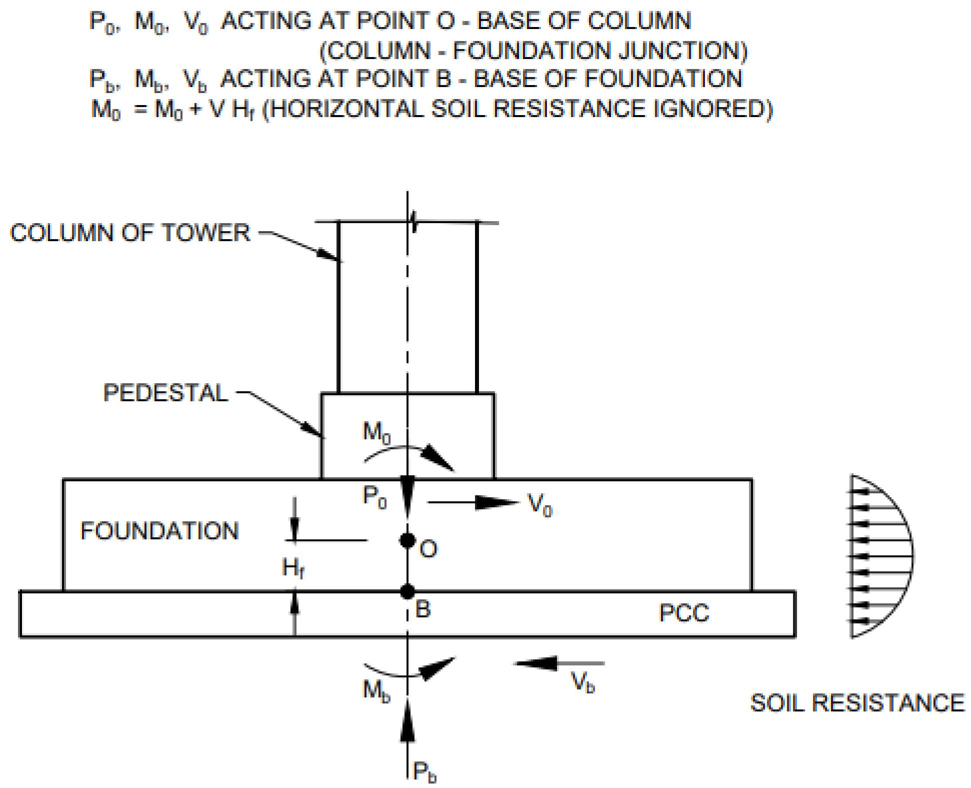


FIG. 12 A

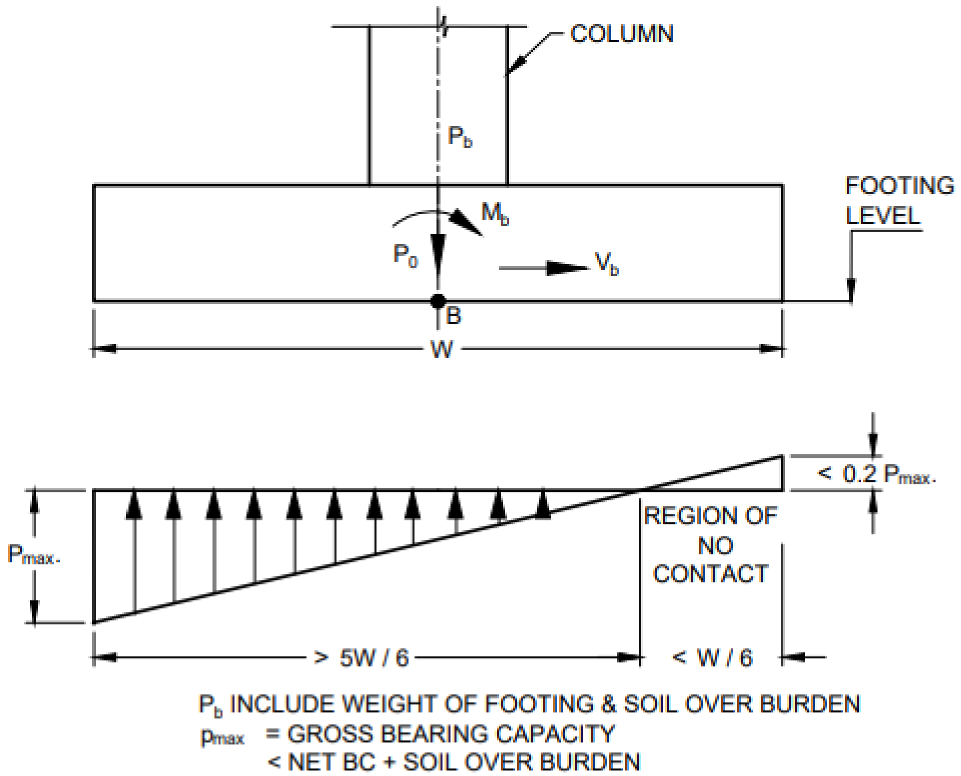


FIG. 12 B

FIG. 12 FOUNDATION PRESSURE.

14.5.3.3 At the contact plane of founding stratum and structural foundation, tension (as no contact), if develops, shall be small. Tension will be considered as small, if it is not more than one fourth of the maximum actual compression for load combinations with maximum wind or maximum earthquake. Tension can be up to one fourth of the maximum actual compression for medium soils having corrected N value (standard penetration) more than 15 or founding strata is very hard like soft rock.

Maximum compression and tension both will take in to account the self-weight of foundation and the soil burden over it. However, after assuring that tension is small, redistribution of compressive stress will be worked out assuming no stress in tension zone. Maximum compressive stress thus estimated should remain within safe bearing pressure, allowing permissible increase in the pressure for wind or earthquake combination.

14.5.3.4 The tension check (as in **14.5.3.3**) shall be applied both for tank full and tank empty cases. For load combinations without wind or earthquake such tensions cannot be permitted, except for foundations on soft rock, or rock (as in **14.5.4**).

14.5.3.5 In all cases, neglecting tension in contact, redistribution of contact stress shall be worked out, and thus computed maximum compression will be governed by net safe bearing pressure of founding stratum enhanced by permissible increase in safe bearing pressure, and such condition shall be taken for checking strength design of foundation.

14.5.3.6 In case of rectangular or square foundation, the maximum bearing pressure at a corner can exceed the permissible pressure as in **14.5.3.3** by an amount which is smaller of the half of increase in pressure from average to maximum at edge along the two principal directions, provided actual maximum bearing pressure is within the ultimate bearing capacity.

14.5.4 More tension (than dealt in **14.5.3.3**) causing loss of contact with the founding stratum under foundation can be allowed if the net allowable bearing pressure of strata is very high (that is, founding strata soft rock or rock), and the foundation is specifically checked for stability against overturning as per **9.1**.

14.5.5 Columns can be provided with pile cap and pile foundations. Group of piles or a pile shall be designed such that a tolerance for error in placing any pile up to 100 mm should be catered. Piles shall be designed for the moments and shear due to horizontal loads. All pile caps shall be connected by braces, which are within ground level.

In case of single pile proposed under a column more rigorous analysis of staging together with piles for horizontal loads, sway and eccentricity of pile due to construction tolerance shall be carried out.

14.6 Access

Access to the tank shall be provided by means of ladder (in steel/aluminium/RC or any other suitable material), or stair with landings adequately tied to the staging. For ladders, safety cages shall be provided as per safety requirements, which shall start at about 3 m height, and also at 2.5 m after each landing.

Spiral staircase carried by a single column may be provided. Stair, if carried by columns other than the columns of staging, shall be braced with minimum two columns of the main staging (or at two places on staging minimum 2 m apart).

14.6.1 For a small tank, a ladder may be preferable rather RC stair or spiral stair, because these RC members can impart significant eccentricity (centre of mass to centre of stiffness) to be considered in dynamic analysis of tank.

14.7 For some staging, provision of a room for office or storage may be required. In such a case the load of masonry wall is to be carried by ground level braces, and the roof (RC slab or sheeting) is to be carried by the brace level above GL.

For earthquake analysis, as a simplification the equivalent weight of such masonry and roof will be added to the DL of container. Equivalent load could be as below:

$$W_{\mathrm{emr}} = (W_{\mathrm{m}} \times {h_{\mathrm{m}}}^3 + W_{\mathrm{r}} \times {h_{\mathrm{r}}}^3)/{h_{\mathrm{c}}}^3$$

where

W_{emr} = Load equivalent to the load of masonry and roof of ground floor room;
W_{m} = dead load of masonry of room;
${h_{\mathrm{m}}}^3$ = height of CG of masonry from base of staging (that is, foundation top);
W_{r} = dead load of roof of room;
h_{m} = height of CG of masonry from base of staging (that is, foundation top),
${h_{\mathrm{r}}}^3$ = height of CG of roof from base of staging; and
${h_{\mathrm{c}}}^3$ = height of CG of container from base of staging.

15 SHAFT TYPE STAGING

15.1 The staging may be in the form of single shaft circular or polygonal in plan and may be tapering (changing radius of centreline). The area enclosed within the shafts may be used for providing the pipes, stairs, pumps, electrical control panels, valves, etc. Recommendations about cylindrical shell of revolution only are given in this standard. For other type of shafts, designer has to take appropriate decision.

15.2 Circular Shaft Staging (Circular Cylindrical Shell)

15.2.1 Minimum thickness of shaft shall be decided by the considerations of constructability, which also depends on the height of formwork for one lift of the wall concrete and requirement of internal vibrator. For shaft having horizontal and vertical steel near to both (inside and outside) faces of shell, the minimum thickness will be the summation of cover on each face, two layers (two curtains) of steel bars on each face (total 4 layers) and the gap in middle governed by the method of compaction of concrete, and the type and size of vibrator to be used. Smaller thickness can be provided for the shaft having vertical and horizontal steel at the middle of the thickness of shaft (that is, having only one curtain/mesh of steel). However, strength capacity of such shaft shall be lower. In no case shaft shall be less than 150 mm thick. It is preferable to have thickness 200 mm or more for proper constructability.

15.2.1.1 Thickness of the shaft shall not be less than that from the guideline given below:

a) For shafts with centre line radius less than 4 000 mm;

$$t_{\mathrm{min}} = 150 + (R_{\mathrm{c}} - 2000)/40 \text{ mm}$$

b) For shafts with centre line radius greater than 4 000 mm,

$$t_{\mathrm{min}} = 200 + (R_{\mathrm{c}} - 4000)/60 \text{ mm}$$

Where R_{c} is centre line radius of shaft in mm, and t_{min} is minimum thickness

15.2.1.2 Additional thickening shall be provided at top and bottom ends of shaft (that is, at junctions with foundation and with container). This is required to account for secondary moments and eccentricities. End thickness shall not be less than 1.25 times the required design thickness of shaft.

15.2.1.3 Additional vertical reinforcement (over the normal vertical reinforcement at section away from end), at the ends of shaft on each face shall not be less than 0.1 percent of the thickness at end.

15.2.2 *Reinforcement*

The percentage reinforcement given in **15.2.2.1** are for deformed bars of Fe 500 or higher grade. For lower grade bars, the minimum percentage of steel shall be inversely proportional to the grade of steel being provided.

The percentage reinforcement given in **15.2.2.2** are for deformed bars of Fe 500 grade. For any other grade of steel bars, the minimum percentage of steel shall be inversely proportional to the grade of steel being provided.

15.2.2.1 *Vertical reinforcement*

The minimum vertical reinforcement shall be 0.25 percent of the horizontal section under consideration. Reinforcement on the exterior face shall be 50 percent or more of the minimum requirement. Reinforcement on the interior face shall not be less than 40 percent of the minimum requirement. Total on the faces shall not be less than the specified reinforcement. The size of vertical bar shall not be more than one-tenth the thickness of wall. Higher size bar may be provided at the centre of thickness. To maintain spacing of reinforcement mesh between the two faces, spacer clips shall be provided (*see* Fig. 13).

Thin shaft may have one mesh of reinforcement at middle, meeting the requirement of minimum steel (0.25 percent). Strength capacity of shaft shall be reduced if it has only one mesh of reinforcement. If factored shear stress demand in wall exceeds 0.25$\sqrt{f_{\mathrm{ck}}}$, the reinforcement in single mesh is not permitted. (*see* also **15.3.1.1**). The diameter of vertical bars shall be minimum 10 mm and the maximum centre-to-centre distance of reinforcement in each layer shall not exceed 300 mm $\mathrm{c/c}$.

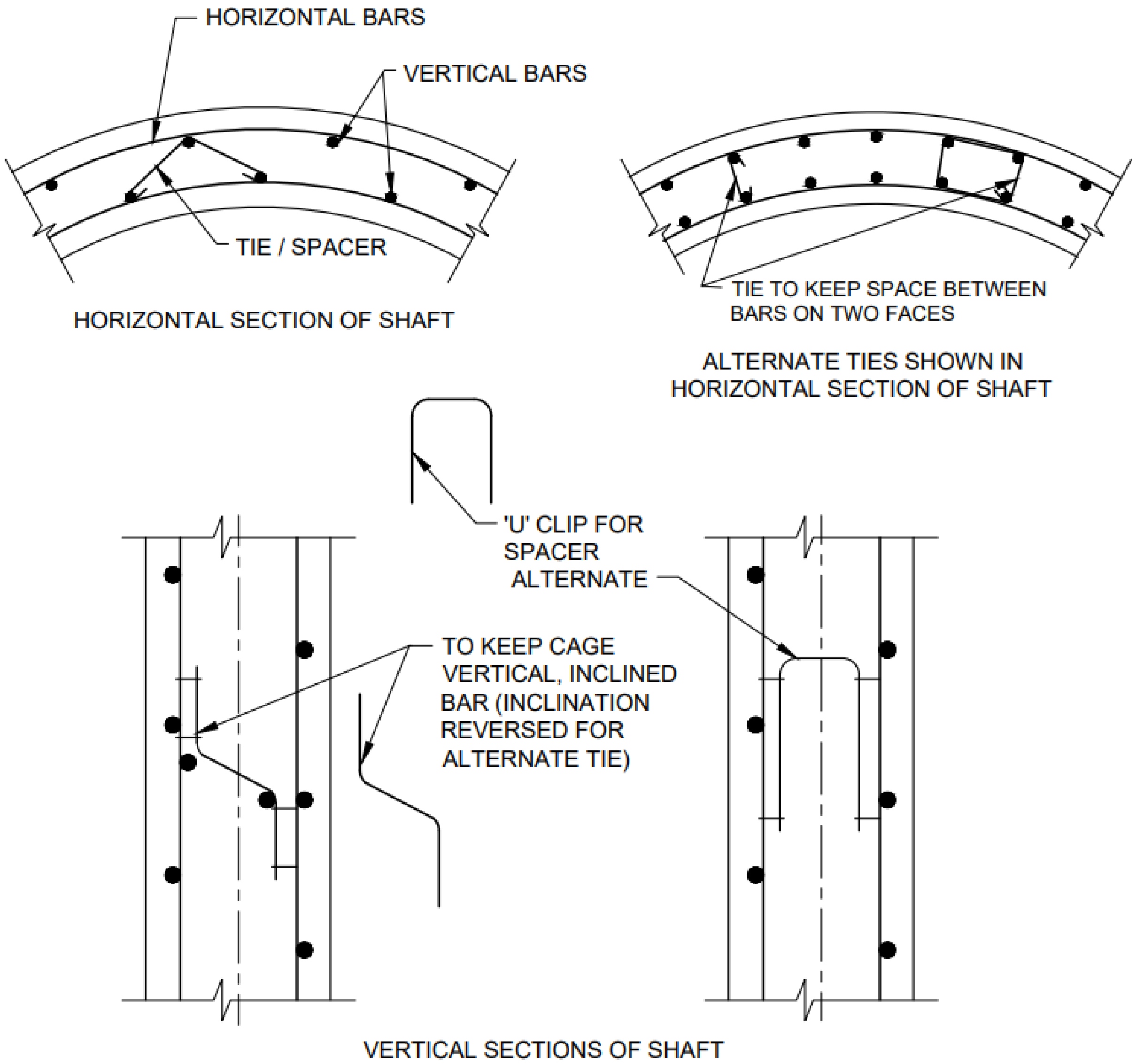


FIG. 13 DETAILS OF TIE/ SPACER IN SHAFT.

15.2.2.2 *Circumferential reinforcement*

The minimum circumferential reinforcement shall be 0.20 percent of the concrete area in vertical section under consideration, subject to a minimum of 400 mm² per meter height. If the vertical reinforcement is provided in two layers, the circumferential reinforcement shall also be provided in two layers and the minimum specified shall be divided nearly equally in each layer. The spacing of bars on a face shall not be more than 1.5 times the shell thickness or 300 mm.

For mesh of bars in two layers, circumferential reinforcement shall be placed nearer the faces of the shell with specified clear cover. Smaller size of horizontal bars are preferred, however size of bar should be such that spacing be not less than 80 mm centre to centre.

Parallel to horizontal construction joint, steel reinforcement shall not be provided within a zone (above or below the horizontal construction joint) as given in Fig 14. The interface of construction joint shall be rough. On outside face extra circumferential bar shall be provided within 50 mm of the construction joint, each at top and bottom.

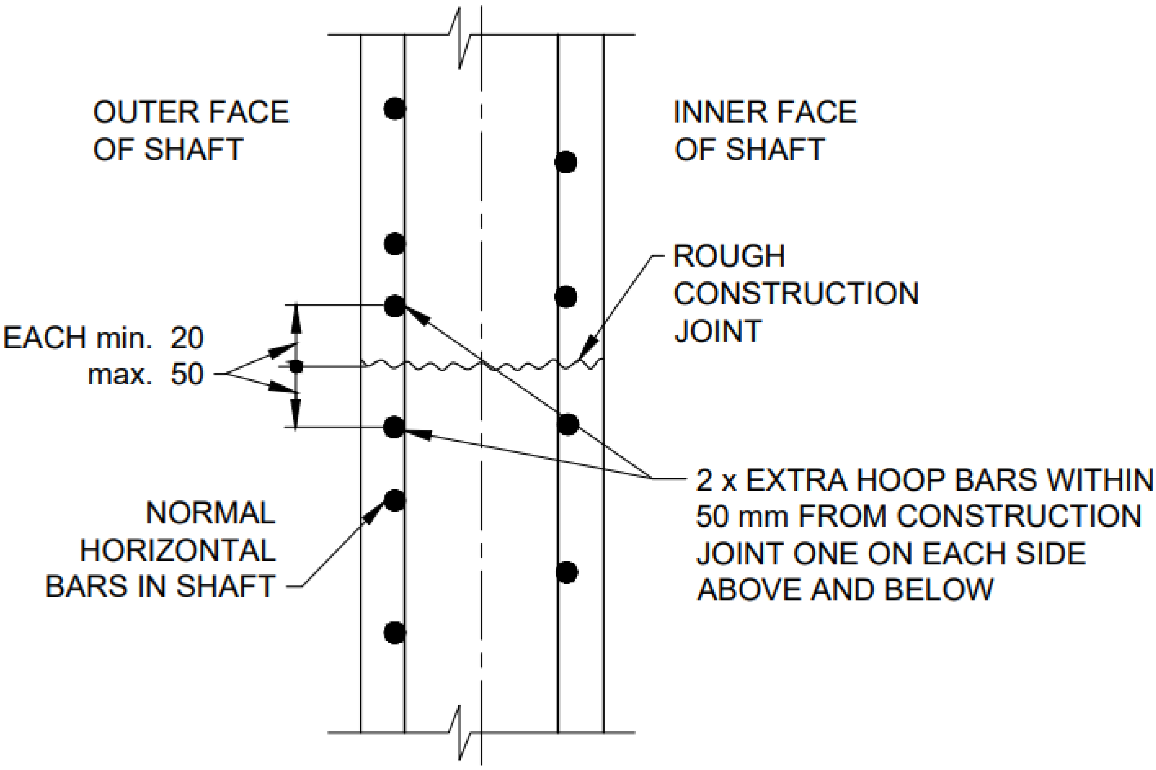


FIG. 14 VERTICAL SECTION OF SHAFT AT CONSTRUCTION JOINT

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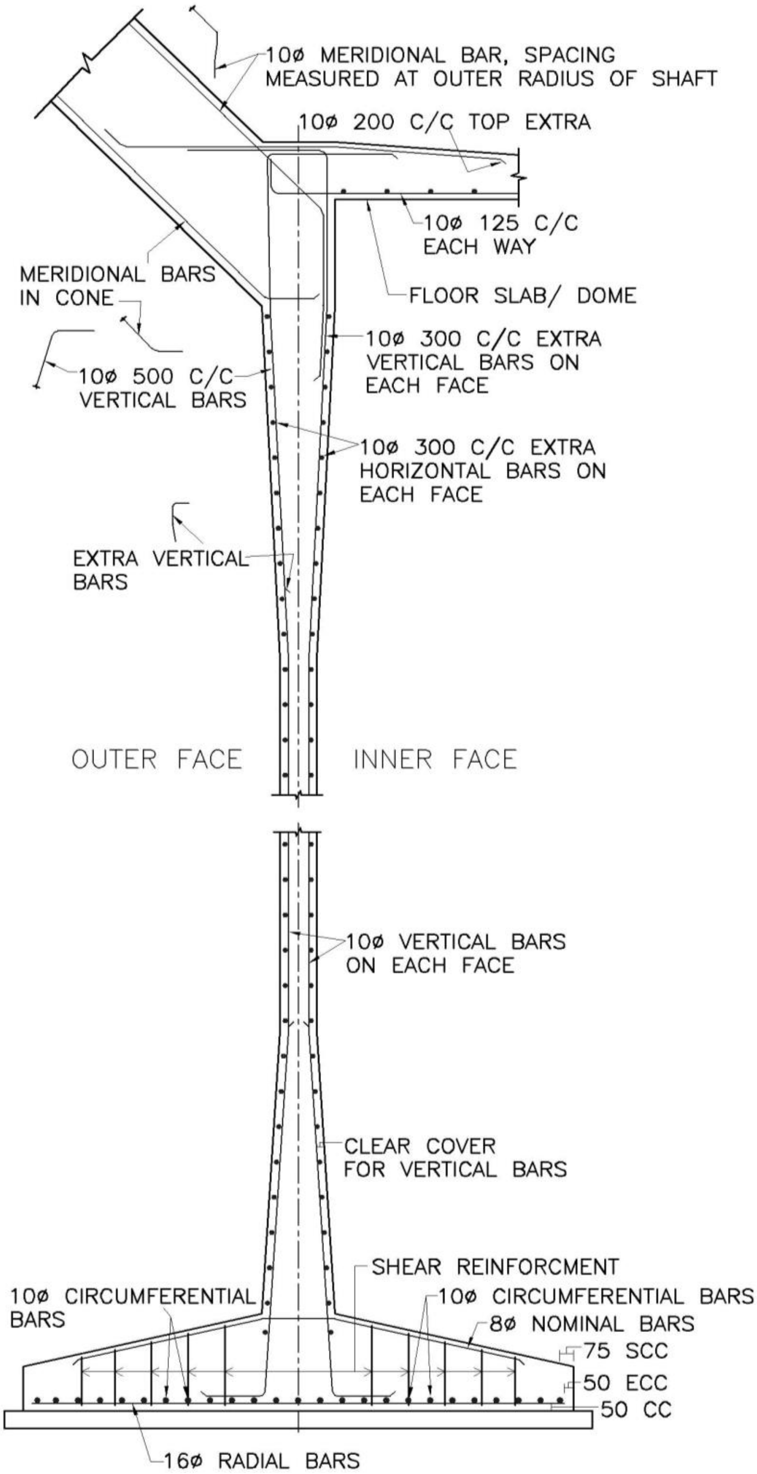
15.2.2.3 The detailing of shaft at the opening shall take into consideration the stress concentration at corner of opening, and provision of effective continuity in the reinforcement above, at the sides and below the opening. Diagonal reinforcement is required to control crack width at corner. The requirement of extra reinforcement should be designed based on the detailed analysis of shaft in the region of opening. Typical reinforcing details are shown in Fig. 15A, and typical details at openings are given in Figs. 15B and C. Such analysis can be by finite element or any other suitable method. In absence of such analysis and design, extra reinforcement shall be provided as given below:

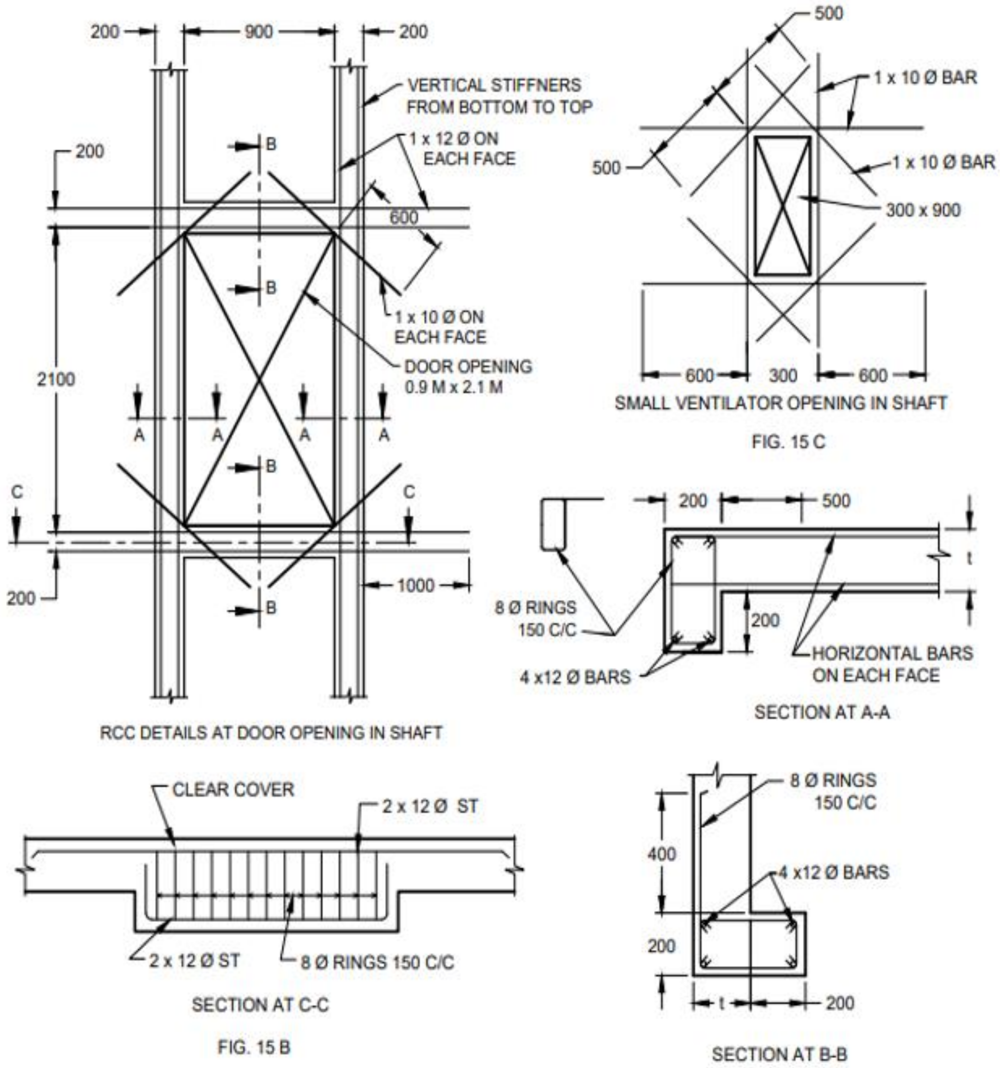
a) At top and bottom of each opening, additional horizontal reinforcement shall be placed having an area at least equal to 70 percent of the area of the calculated design circumferential reinforcement interrupted by the opening, and shall extend beyond the opening to a sufficient distance as development length of bar plus 300 mm. This reinforcement shall be placed within a height not exceeding concrete thickness of shaft at opening.
b) At both sides of each opening, additional vertical reinforcement shall be placed having an area at least equal to 60 percent of the area of the established design vertical reinforcement interrupted by the opening, and shall extend beyond the opening by development length. Size of extra vertical bars shall be not less than 12 mm. If the vertical height of opening is more than 8t_s (‘t_s’ is thickness

of shaft in mm), vertical bars (minimum 4 in numbers) at the edge of opening shall have lateral ties as per Clause **26.5.3.2** of IS 456. The extra vertical steel shall not be less than 0.8 percent of $2t_s{}^2$; and

c) At the corners of rectangular opening, extra diagonal reinforcement with cross-sectional area not less than 30 percent of the total extra horizontal and vertical required shall be placed at each corner of the opening. The diagonal bars shall extend from corner by development length $+ 300$ mm.



15(a) SECTION OF SHAFT WITH ANNULAR RAFT FOUNDATION



15(b)

FIG. 15 OPENING DETAILS IN SHAFT

15.2.2.4 If the height of opening in shaft is more than $12t_s$, the vertical edge of the shaft at opening will require a detailed check for the compressive stresses. At the edge of opening stiffener may be provided, which may project on any one face of shaft.

15.2.2.5 For opening less than 0.6 m width (horizontally) the diagonal reinforcement may be half the value recommended above in **15.2.2.3** (c).

15.2.2.6 For openings smaller than 2 times the thickness of shaft, only nominal extra vertical and horizontal steel can be provided.

15.2.3 The minimum clear concrete cover over the horizontal reinforcement typically can be 35 mm for the outer face and 25 mm for the inner face of the shaft. More clear cover could be specified for the exposure condition being considered, as per requirement of IS 456.

15.2.4 Vertical reinforcement across a horizontal construction joint (per m) shall not be less than A_{st} as below for each 1 m length of horizontal section of wall:

$$A_{\mathrm{st}}/(1\,000\,t_{\mathrm{s}}) = 0.92\,\{t_{\mathrm{v}} - P_{\mathrm{u}}/(1\,000\,t_{\mathrm{s}})\}/f_{\mathrm{y}}$$

where

t_{v}= factored shear stress at the joint (N/mm²)
P_{u} = factored axial load per m (N); and
f_{y} = characteristic strength of steel (N/mm²);

15.2.4.1 Additional vertical dowel bars may be designed at the construction joint if required, assuming 20 percent reduction in compressive strength capacity due to construction joint.

15.3 Analysis for Circular Shaft

15.3.1 *Bending Moment (BM) due to Ovalling*

Under wind load, due to ovalling of horizontal slice of shaft as ring, bending moment will develop causing horizontal tension at one face and compression at other face of shaft. In absence of analysis, the ring moment (in Nm/m height of shaft) can be assumed as below:

$$M_{\mathrm{oe}} = M_{\mathrm{oi}} = 0.33\,p_{\mathrm{d}} C_{\mathrm{pe}} R_{\mathrm{c}}^{2}$$

where,

M_{oe} = BM due to ovalling causing tension at external face in Nm/m;
M_{oi} = BM due to ovalling causing tension at internal face in Nm/m;
p_{d} = Design wind pressure at the level under consideration in N/m^2; and
C_{pe} = Pressure coefficient 1.7 to 2.0, as per IS 875 (Part 3);
R_{c} = Mean radius of ring of the concrete shell in m.

Bending moment due to ovalling may be considered negligible at a diaphragm (or horizontal stiffener) if provided, and also at the ends of shaft (top at container and bottom at foundation if rigidly connected). This moment may be considered to be vary linearly to the design value at a distance $3\sqrt{(t_{\mathrm{s}}.R_{\mathrm{c}})}$ from rigid end or the stiffener.

In most cases, shaft may be deemed to be safe for the effect of hoop force and shear stress due to ovalling if designed for the bending moment.

15.3.1.1 As long as flexural tension due to the ring moment at service load is within $0.25\sqrt{f_{\mathrm{ck}}}$, horizontal reinforcement need not be designed for the bending moment, and only minimum reinforcement may be provided. If flexural tension is within $0.15\sqrt{f_{\mathrm{ck}}}$, horizontal reinforcement can be provided at the middle of shaft thickness.

If the flexural stress exceeds the limit, reinforcement shall be designed and provided in two layers, ignoring the flexural tensile strength of concrete. Minimum horizontal steel as provided can be accounted to resist the ovalling BM.

15.3.2 *Vertical Stress due to Horizontal (Wind/E*arthquake*) Load on Elevated Tank*

For shaft as cylindrical shell, membrane stresses in vertical direction (meridional) shall be estimated. Membrane stresses in vertical direction (meridional) shall be the combination of stress due to gravity load and that due to cantilever bending of shaft as a whole.

Vertical stress as calculated (in **15.3.3.1, 15.3.3.2, 15.3.2.3**) shall be considered as direct compressive (membrane) stress in the shell. These equations are based on linear elastic behaviour. For limit state design method, results of these equations can also be used with appropriate load factors; alternately equations or design-aids derived for limit state design with assumptions as given in **15.3.2.1, 15.3.2.2** and **15.3.2.3** shall be used.

15.3.2.1 For the purpose of design, total membrane stress shall be considered as direct compression and not as bending compression. Bending stress in the shell may occur due to bending moment developed as a result of construction defects (such as out of alignment) and errors, differential wind pressure (gust) on the surface of shaft, local ovalling or distortion of shaft, etc. (*see* **15.3.3.5**).

15.3.2.2 For limit state design following assumptions shall apply:

a) For meridional stress, the maximum compressive strain in concrete as axial compression shall be limited to 0.002; and
b) For combined meridional compression and local bending in the shell, the maximum compressive strain at the highly compressed extreme fibre in concrete when there is no tension on the section shall be 0.003 5 minus 0.75 times the strain at the least compressed extreme fibre.

15.3.2.3 While calculating the resistance to vertical stress, the contribution of steel in compression shall be effective only if vertical steel bars are laterally tied conforming to all requirements (including transverse reinforcement) of IS 456 Clause **26.5.3**; or else, the contribution of steel in compression shall be neglected.

15.3.3 *Equations for Vertical Stress*

15.3.3.1 The whole section is under compression (for serviceability limit state):

a) for annular sections, if

$$\frac{e}{R_c} \leq \frac{1}{2} \quad \text{. . . . . (1)}$$

In such cases the maximum vertical compressive stress in concrete is given by:

$$\sigma_{cv} = \frac{W}{2\pi R_c t_s}\left(1 + \frac{2e}{R_c}\right) \quad \text{. . . . . (2)}$$

b) for annular section with one opening:

$$\frac{e}{R_c} \leq \frac{1}{2(\pi-\beta)}\left[\frac{(\pi-\beta)^2 - sin^2\beta}{(\pi-\beta)cos\phi + sin\beta} - 3sin\beta\right] \quad \text{. . . . . (3)}$$

then in such cases, the maximum vertical compressive stress is given by:

$$\sigma_{cv} = \frac{W}{2(\pi-\beta)R_c t_s}\left[1 + \frac{2\left\{\frac{e}{R_c}+\frac{sin\beta}{(\pi-\beta)}\right\}\{(\pi-\beta)cos\beta+sin\beta\}}{(\pi-\beta)-\frac{1}{2}sin2\beta-\frac{2sin^2\beta}{(\pi-\beta)}}\right] \quad \text{. . . . . (4)}$$

where

$$e = \frac{M}{W} = \frac{Moment\ in\ vertical\ plane\ at\ the\ section\ under\ consideration\ in\ Nmm}{Total\ Vertical\ load\ above\ section\ under\ consideration\ in\ N}$$

σ_{cv} = vertical stress in concrete at centre line of shaft (point farthest from bending axis) in N/mm^2.
R_c = Mean radius of circular section under consideration in mm;
t_s = thickness of shaft shell at section under consideration in mm;
β = Half the angle subtended by the neutral axis as a chord on the circle of radius r, in radians unless otherwise specified; and

15.3.3.2 If e/R_c is greater than the corresponding right hand side of expressions (1) or (3), then α, defining the position of neutral axis, may be calculated from the general expression (5) by trial (*see* Fig. 16).

Put $\beta = 0$, for annular section without opening,

where

α = one half the central angle subtended by neutral axis as a chord on the circle of radius R_c, in radians unless otherwise specified;

$$\frac{e}{R_c} = \left(\frac{A}{B}\right) \quad \text{. . . . . (5)}$$

$$A = \frac{1}{2}(1-\rho)\,(\alpha - sin\alpha - cos\alpha) - \frac{1}{2}(1-\rho+m\rho)\,(\beta + sin\beta + cos\beta - 2cos\alpha\ sin\beta) + \frac{1}{2}m\pi\rho; \text{ and}$$

$$B = (1-\rho)\,(sin\alpha - \alpha\,cos\alpha) - (1-\rho+m\rho)\,(sin\beta - \beta\,cos\alpha) - m\,\pi\,\rho\,cos\alpha$$

m = modular ratio; and
ρ = ratio of area of effective vertical reinforcement to gross area of concrete of shell at section under consideration.

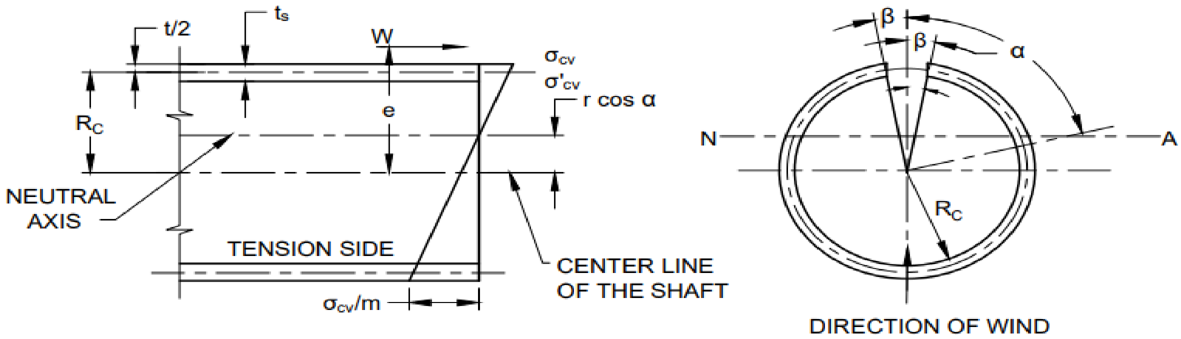


FIG. 16 SHAFT WITH OPENING

15.3.3.3 The maximum vertical compressive stress in concrete due to combined effect of vertical loads and lateral wind loads, σ_{cv} may be calculated by:

$$\sigma_{\mathrm{cv}} = \sigma_{\mathrm{cv}} \left[1 + \frac{t_{\mathrm{s}}}{2R_{\mathrm{c}} \cos\beta(\cos\beta - \cos\alpha)}\right] \quad \ldots\ldots\ldots (6)$$

where

$$\sigma_{\mathrm{cv}} = \frac{W}{2R_{\mathrm{c}}t_{\mathrm{s}}}\left[\frac{(\cos\beta-\cos\alpha)}{(1-\rho)(\sin\alpha-\alpha\cos\alpha)-(1-\rho+mp)(\sin\beta-\beta\cos\alpha)-m\rho\pi\cos\alpha}\right] \quad \ldots\ldots\ldots (7)$$

15.3.3.4 Maximum vertical tension in shell (at centre line of thickness) may be calculated as follows:

a) *At centre line of thickness* —

$\sigma_{\mathrm{sv}} = m\,\sigma_{\mathrm{cv}}(1 + \cos\alpha) / (1 - \cos\alpha)$; and

b) *at extreme fibre* —

$$\sigma_{\mathrm{tv}} = \sigma_{\mathrm{cv}}(1 + \cos\alpha + 0.5\, t_{\mathrm{s}}/R_{\mathrm{c}}) / (1 - \cos\alpha)$$

15.3.3.5 In addition to vertical membrane stress estimated, vertical bending stress will also occur in the shell. Vertical bending moment will develop due to local ovalling and bulging effect and also due to eccentricity (out of geometry for middle surface of shaft) achieved and variation in stiffness of shell in construction and few other irregularities.

NOTE – Vertical stress due to bending of staging as a whole, gives membrane (meridional) stress and not flexural. The local bending will give stress as compression on one face of shaft and tension on other face that is, within the thickness of shaft stress will vary from compression to tension. Neglecting such vertical bending stresses requires higher factor of safety with the vertical membrane stress.

15.3.3.6 In addition to vertical membrane force, shaft shall also be designed for local bending moment which may be due to deviation of middle surface of the cylinder as constructed (that is, eccentricity at a point or variation in curvature) or wind pressure variation etc. In absence of a detailed estimation to assess vertical bending moment, a minimum M_{v} shall be assumed as below.

$M_{\mathrm{v}} = ecc_{\mathrm{m}}\sigma_{\mathrm{cv}}t_{\mathrm{s}}$ (in Nmm/mm); and
$ecc_{\mathrm{m}} = R_{\mathrm{c}}/500$ mm, or $t_{\mathrm{s}}/30$, or 5 mm whichever m.

where,
σ_{cv} is maximum meridional compression in N/mm², and t_{s} in mm.

NOTE – For Limit state design, eccentricity moment shall be multiplied by partial load factor 1.5.

15.4 Reduction of Strength Capacity

The strength capacity of concrete will be reduced by multiplication factors as below for vertical stresses.

15.4.1 If vertical and transverse reinforcement conform to all requirements of IS 456 as for lateral ties in columns, the capacity reduction factor C_{rd} (for detailing) shall be 1.0, or else it will be 0.75. If lateral ties are not provided to vertical bars as per requirement of column, the contribution of vertical steel in compression zone shall also be neglected.

15.4.2 Based on constructability, capacity reduction factor, C_{rt} will be as below:

a) Shaft having steel on both faces: $C_{\mathrm{rt}} = t_{\mathrm{s}}/200$, $C_{\mathrm{rt}} \leq 1.0$; and

b) Shaft having steel in one layer in middle: $C_{\mathrm{rt}} = 0.15 + t_{\mathrm{s}}/400$; $0.4 \leq C_{\mathrm{rt}} \leq 0.6$

where, t_{s} = thickness of shaft in mm

15.4.3 Slenderness of wall expressed as $C_{\mathrm{rs}} = 40\ t_{\mathrm{s}}/R_{\mathrm{c}}$, shall not be greater than 1.0. At construction joint, C_{rs} may be taken equal to 1.0, if the joint strength reduction is done by 20 percent.

15.4.4 Capacity reduction factor $C_{\mathrm{r}} = C_{\mathrm{rd}} \times C_{\mathrm{rt}} \times C_{\mathrm{rs}}$

15.5 Limit State Design Method

For limit state design, proper value of W (membrane forces) and BM (bending moments, etc.) after multiplying with the appropriate load factor shall be accounted.

Formulae give in **15.3.2.1**, **15.3.2.2**, **15.3.2.3** may be used to assess the vertical meridian force as axial load. Section of shaft is to be designed for axial load and bending moment. Capacity reduction factors as in **15.4.4** shall also be applicable.

15.6 Eccentricity of Container

For shaft and foundation, eccentricity may occur if the:

a) tank is not concentric with the support shaft;
b) support wall is out-of-plumb; or
c) foundation tilts because of differential settlement.

Shaft shall be designed for an eccentricity of container not less than the value given below:

$$e_{\mathrm{g}} = 0.002\,5 \times (\text{staging height} + R_{\mathrm{c}})$$

Above eccentricity is minimum and may be assumed to include the allowance for angular distortion of foundation and eccentricity due to error in construction, etc.

Additional bending moment at base of shaft shall be equal to that given below :

(Vertical loads from container in a wind/earthquake combination + half the DL of staging) $\times\ e_{\mathrm{g}}$

This BM will be additive to the BM due to horizontal load (wind/earthquake). While this eccentricity effect is considered, classical $P - \Delta$ can be neglected.

As the above eccentricity is accounting many factors, it should not be taken as maximum tolerance in construction. Higher construction tolerance will induce significant local bending stresses in shaft, which will very much affect the critical buckling load.

15.7 Eccentricity due to Settlement

The section of the shaft shall also be checked for stresses resulting from the possible differential settlement of foundation as per IS 1904.

15.8 Polygonal Shaft

Polygonal shaft may be designed as a circular shaft considering equivalent radius of inscribed circle for the middle surface of the shaft, if it is a regular polygon having each side not more than 12 times the thickness of shaft. For thinner and larger shafts detailed analysis for bending moments and stability shall be carried out.

15.9 Shaft should be provided with opening for door entry, and for ventilation and natural light. Vertical stiffeners may be provided on the sides of opening.

15.9.1 If inside the shaft, earth filling is higher than the ground outside, the shaft shall be designed for the hoop force developed due to soil pressure from inside assuming 0.5 m soil on outside may degrade.

15.10 Foundation

15.10.1 Foundation can be of any one of the types that is, strip, annular mat, ring mat, ring raft, full mat or full raft.

Mat design assumes nearly uniform or gradually varying pressure (without consideration of soil stiffness), and in raft design stiffness of foundation soil is considered (that is, soil-structure interaction). Slab with central opening is viewed as ring or annular foundation. Shell foundation may be provided.

For design of foundation as mat, the soil pressures are not related to the vertical deflections of foundation. Foundation slab can be designed assuming it as a mat (that is, designed for assumed uniform or linearly varying soil reaction). For hard, stiff and medium soils such an approach can be made. For soft soils the mat approach requires more materials compared to raft approach, but are permissible being safe.

For raft design relative stiffness of soil and structural foundation determines the soil pressure taking account of modulus of sub grade reaction. Raft design gives more economy (compared to mat) for the smaller bearing capacities, though analysis is complicated.

Reference can be made to IS 11089.

Most equations available are for uniform thickness of foundation slab, however as per practice higher thickness may be provided near the faces of shaft. Such a deviation is normally permissible, while analysis is for uniform thickness.

All mats or raft foundation shall be checked for safety against shear failure.

15.10.2 For foundation soil stiff or hard, annular foundation of shaft can be designed as radial strip with following assumption:

a) If the additional pressure on foundation due to bending of tower is within 30 percent of the pressure due to vertical loads only, this simplified method is applicable.

b) Each radial strip will be treated as rigid element having centre of gravity of its area at the radius of centre of shaft. Following equations can be used. This approach is applicable till $r_{\mathrm{o}}/r_{\mathrm{c}} < 1.45$. In this simplification, the circumferential moments are neglected.

Area of foundation = $p\ (r_{\mathrm{o}}{}^2 - r_{\mathrm{i}}{}^2)$

CG of radial strip at radius = $(r_{\mathrm{o}}{}^2 + r_{\mathrm{o}} r_{\mathrm{i}} + r_{\mathrm{i}}{}^2)$ / $(1.5\ (r_{\mathrm{o}} + r_{\mathrm{i}}) \approx R_{\mathrm{c}}$

Outer projection from face of shaft will be at radius $r_{\mathrm{c}} + t_{\mathrm{f}}/2$ to r_{o}

At outer face of shaft BM due to outer projection (in Nm/m) =

$$p\ (r_{\mathrm{o}} - R_{\mathrm{c}} + t_{\mathrm{f}}/2) \times [(r_{\mathrm{o}} + R_{\mathrm{c}} + t_{\mathrm{f}}/2)/6\ -\ r_{\mathrm{o}}{}^2/\{3(R_{\mathrm{c}} + t_{\mathrm{f}}/2)\}]$$

Inner projection from face of shaft will be at radius $r_{\mathrm{c}} - t_{\mathrm{f}}/2$ to r_{i}

At inner face of shaft BM due to inner projection (in Nm/m) =

$$p\ (R_{\mathrm{c}} - t_{\mathrm{f}}/2 - r_{\mathrm{i}}) \times [(r_{\mathrm{i}} + R_{\mathrm{c}} - t_{\mathrm{f}}/2)/6 - r_{\mathrm{i}}{}^2/\ \{3(r_{\mathrm{c}} - t_{\mathrm{f}}/2)\}]$$

where,

p = uniform design pressure causing bending of foundation (N/m²);
r_{o} = outer radius of strip foundation (m);
r_{i} = inner radius of strip foundation (m); and
t_{f} = thickness of shaft at top of foundation (m).

15.10.3 For a circular mat (full raft) foundation, maximum eccentricity up to

a) 0.25 times the centreline diameter of shaft at base, and
b) 0.20 times the outer diameter of foundation,
Whichever, is higher, may be permitted.

However, neglecting tensile stress in contact, redistribution of stress shall be worked out, thus the maximum compression will be governed by permissible safe bearing pressure of founding stratum, and such condition shall be taken for strength design.

16 DETAILING

16.1 Detailing of reinforcement shall conform to the requirements of IS 456. Staging shall be designed as special moment resisting frame (ductile frame), and the detailing shall conform to the requirements of IS 13920.

16.1.1 If the staging is designed as ordinary moment resisting frame, the column beam junction shall be provided with spacing of lateral ties 8 φ not more than 100 mm c/c, and for square or rectangular column 8 φ at 75 mm c/c.

16.1.2 The column beam junction shall be enclosed in the rings at a spacing not exceeding 100 mm c/c. Where column size (width) is more than brace width, the column rings will continue in the junction; and if width of brace is more than lateral size of column, the rings of the brace shall continue through the junction.

16.2 Within 'T' or 'L' junctions, diagonal tension is produced. In these junctions the tension bars changes its direction by right angle. The tension reinforcement shall be well anchored beyond the position of maximum stress, which is nearly at middle of the curved portion of the bar. (*see* Figs. 17 to 21 and Fig 3 in IS 3370 (Part 2), amendment 1.).

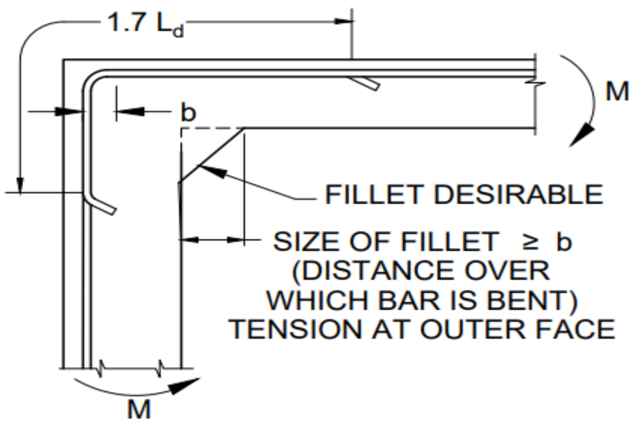


FIG. 17 RIGHT- ANGLED CLOSING JOINT.

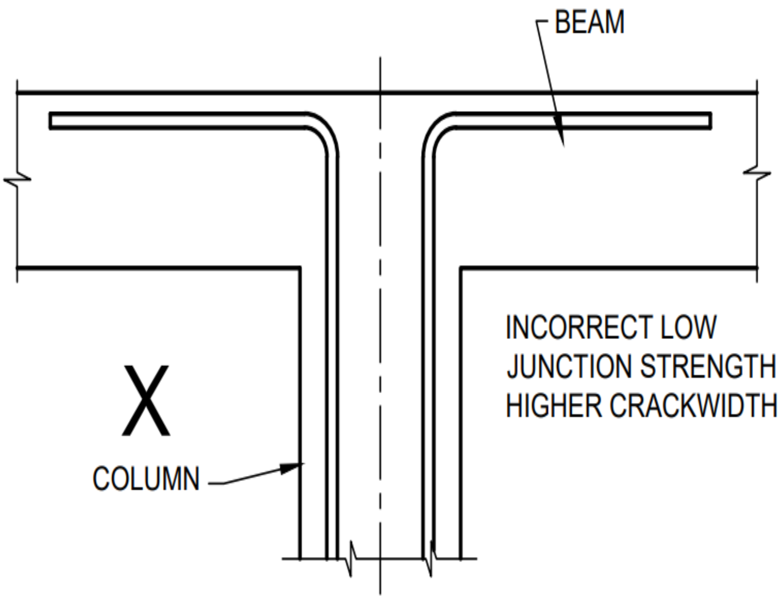


FIG. 18A 'T' JUNCTION (INCORRECT)

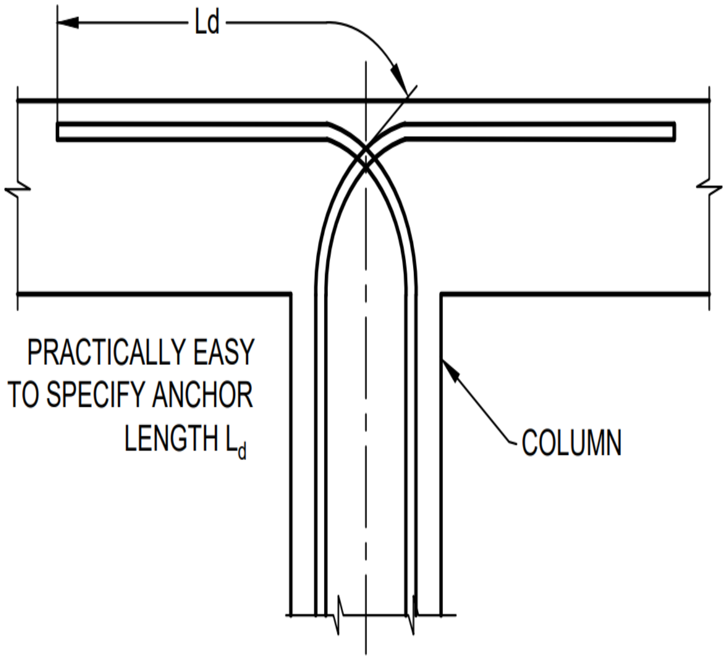


FIG. 18B 'T' JUNCTION (CORRECT)

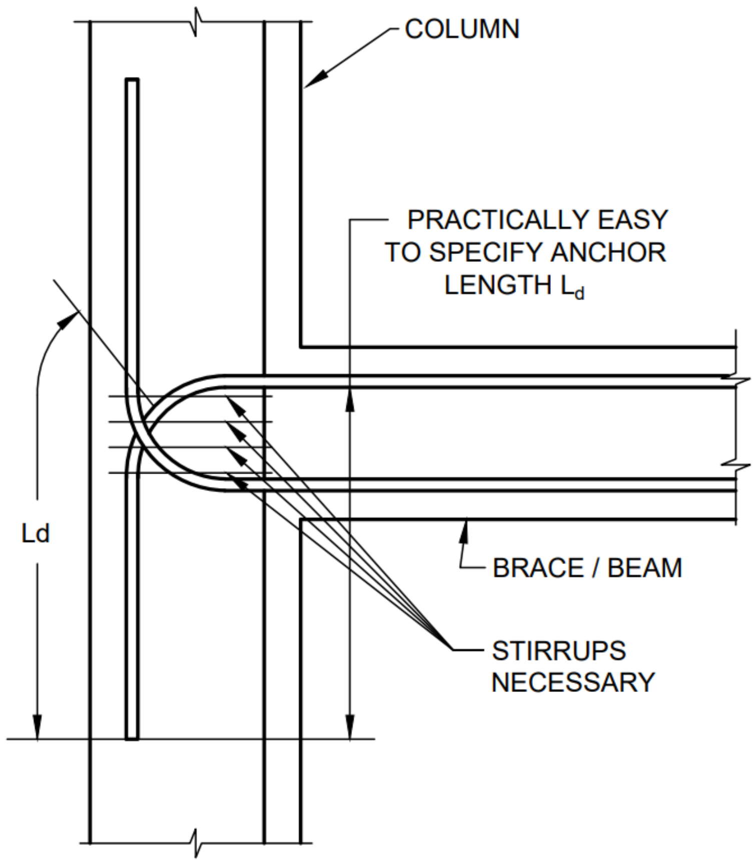


FIG. 19 REINFORCEMENT DETAILING IN 'T' JUNCTION

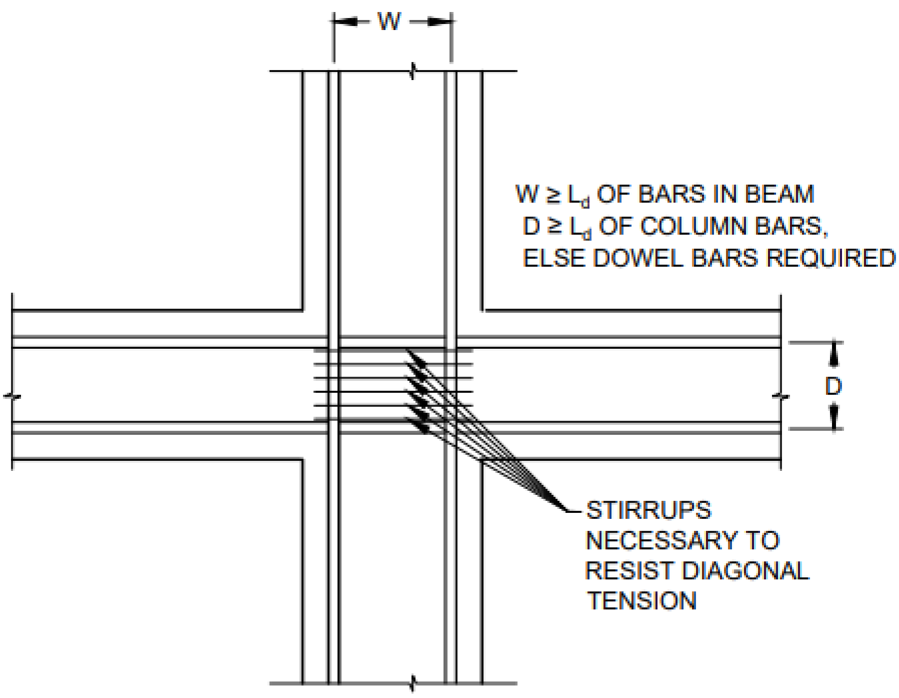


FIG. 20 COLUMN – BRACING CROSS JUNCTION

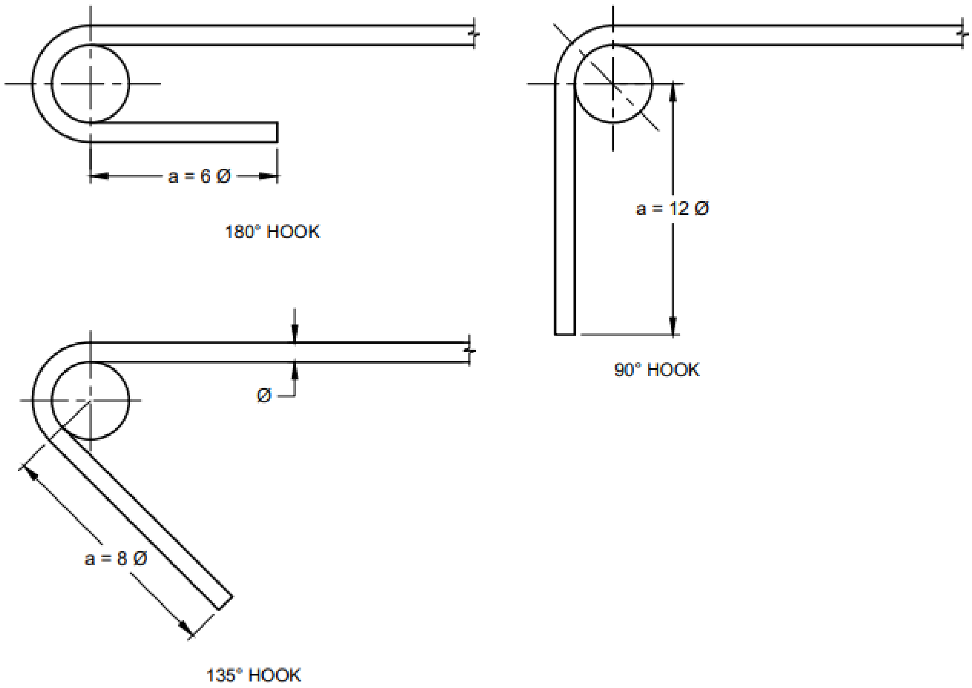


FIG. 21 HOOKS FOR RINGS OR STIRRUPS.

16.3 Minimum reinforcement ratio in brace, at each support ends, shall be ρ_{min}

$$\rho_{\text{min}} = 0.24 \frac{\sqrt{f_{\text{ck}}}}{f_{\text{y}}}$$

The minimum longitudinal top steel shall be ρ_{Min} times the product of width and effective depth. At the bottom face minimum steel can be $0.85 \times \rho_{\text{Min}}$.
Above steel shall be available near each support, for a length 2d or 20 percent of clear span, whichever is more.

16.3.1 Longitudinal steel ratio on any face at any section shall not be more than 2.5 percent.

16.4 Typical reinforcement details for column brace junction are given in Fig. 22.

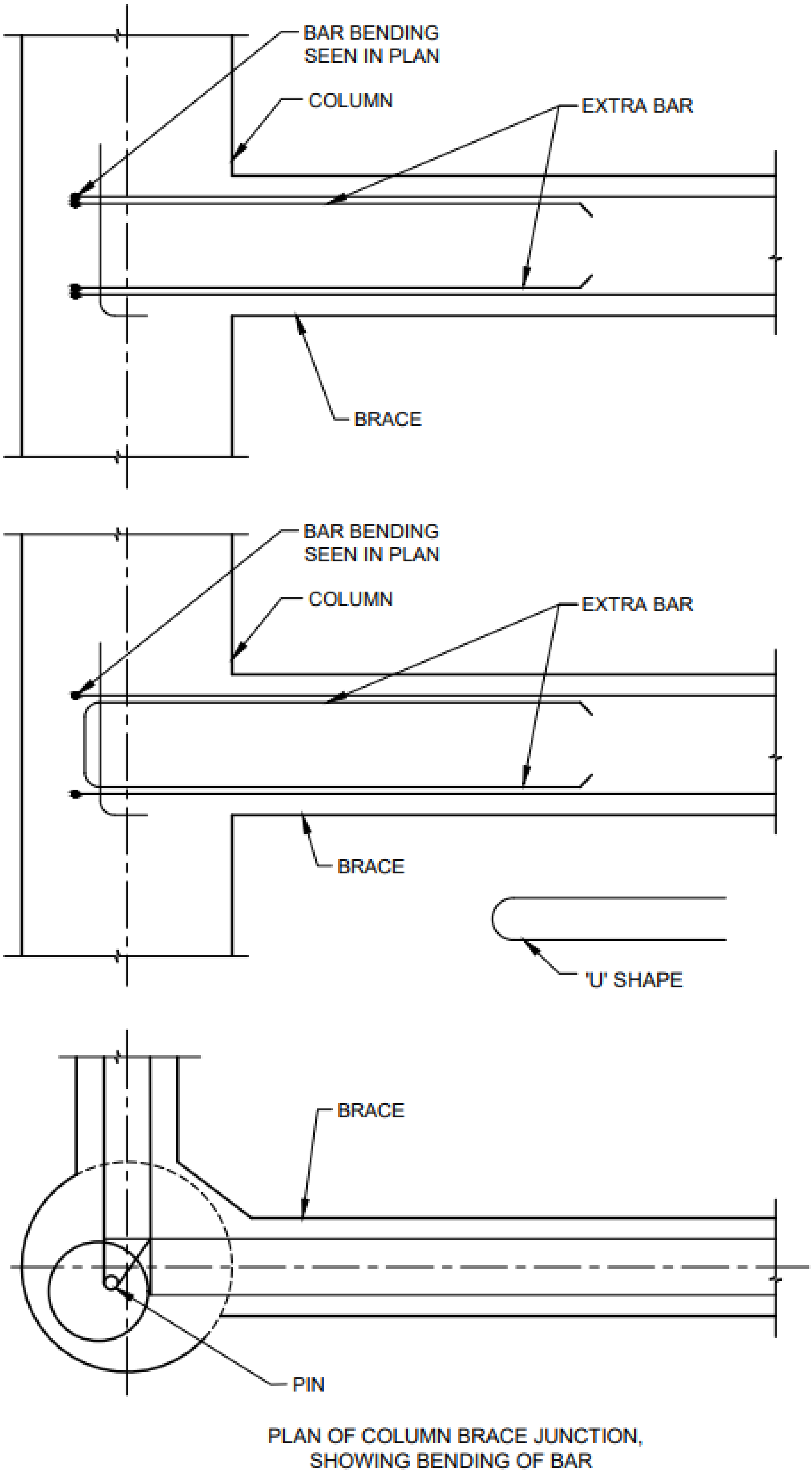


FIG. 22 COLUMN-BRACING JUNCTION

16.5 Galvanic Corrosion

Dissimilar metals shall be electrically isolated to prevent galvanic corrosion.

17 CONSTRUCTION REQUIREMENTS

Construction shall be proper to achieve high level of quality for better reliability and approach the behaviour assumed in design. Guidelines given below are not exhaustive. The designers and construction engineers are expected to have competence to take adequate measures to ensure required structural performance.

Some of the requirements of minimum size, constructability and tolerance have been provided in the main body of the standard. Additional requirements are given below.

17.1 At all construction joints, at the interface, roughness shall be imparted to the concrete placed earlier by removing some mortar portion from the surface. The amplitude of roughness should be of the order of 3 to 5 mm. The interface should be rough and should not have loose material, mortar layer, laitance, etc when fresh concrete is being deposited. [For more details *refer* IS 3370 (Part 1)].

17.1.1 In columns, construction joints shall be avoided in special confining zone (that is, near brace-column junction). Otherwise higher confinement reinforcement shall be provided near construction joints to compensate for lower strength at construction joint. In absence of any estimate and calculation for the same, one set of 12 mm diameter ties shall be provided on either side, within 50 mm of the joint. For circular column 10 mm diameter tie can be used. Location of construction joint shall be marked on the all joints in column should be pressure grouted.

17.1.2 Splices in the column bars within the junction with brace and the special confining zones, shall not be permitted normally; and slices can be provided in other zone say in middle portion of column height. Normally laps should be staggered such that not more than half of longitudinal bars are lapped simultaneously, and lateral ties are spaced at not more than 150 mm c/c at the laps.

17.1.3 If all bars of a column are spliced simultaneously, without staggering, but not within confining zone, the lap length shall be enhanced by 50 percent over the lap length in tension, and lateral ties in form of spiral shall be spaced at not more than 100 mm c/c over the distance of lap length plus 75 mm beyond the both ends of lap. In such laps, longitudinal bars shall not be provided with kink at or near the lap.

17.1.4 For smaller height of column lifts in which contra-flexure does not occur, laps shall be avoided.

17.1.5 Formwork at brace-column junction should be designed and fabricated. It shall be rigid enough to avoid bulging as well as prevent leakage of slurry from plastic concrete.

17.1.6 All brace-column junctions shall be designed and detailed, so as to avoid failure within junction and allow the ductility in the whole frame. Tension within the junction shall be resisted by adequate reinforcement. The drawing shall show detailing at each typical junction.

17.2 Shaft Staging

17.2.1 The wall of shaft is subjected to large compressive forces and generally requires a high degree of accuracy with regard to shell tolerance. Properly designed climbing forms or jump forms with through ties with proper workmanship and checks can achieve the required tolerances. Vertical alignment should be controlled with precise optical or other instruments. Wall forms shall be designed for the full concrete pressure due to concrete depth equal to form height. Thus overloading and excessive deflection can be avoided if forms are filled to full height in a small time.

17.2.2 *Vertical Alignment*

The centre point of shaft shall not vary from its vertical axis by more than 0.2 percent of shaft height.

17.2.3 Over any height of 1.6 m, wall of shaft shall not be out of plumb by more than 10 mm, and over any 10 m height not more than 25 mm.

17.2.4 *Shaft Radius*

The measured centreline radius of shaft at any location shall not vary from the specified radius by more than 10 mm plus 0.1 percent of the specified radius.

17.2.5 *Shaft thickness*

The measured wall thickness shall not vary from the specified wall thickness by more than -5 mm or $+15$ mm. However, average of eight or more readings at a level shall have no negative tolerance.

17.2.6 Laps shall be staggered such that not more than one-third of the bars shall be spliced at any section. For vertical bars lap length shall be the development length in tension. For circumferential bars, lap length shall be two times development length in tension.

17.2.7 At the construction joint, the interface surface of old and new concrete should be rough and should not have loose material, mortar later, laitance etc. Segregation and honey-comb should be avoided at the joint. All construction joints should be grouted to compensate for the loss of strength and stiffness due to interface formed.

17.3 Selected excavated material shall be backfilled by placing and compacting in uniform horizontal layers. Method of compaction shall be such that any damage to the ground brace, columns or shaft is avoided, as well as the fill near these members surface is properly compacted. Similar precautions are required for plinth filling.

On the two sides of shaft, filling should be placed in layers such that any time difference of soil top should not be more than $2 \times$ thickness of shaft, to avoid significant differential earth pressure. Fill on outside can be little higher than inner fill, till ground level, after which plinth inside shall be taken.

17.3.1 Site grading around the tank should provide positive drainage away from the tank to minimize percolation of water in to and prevent ponding of water in the foundation area.

17.3.2 Concrete members below and near ground may need additional protection if the ground conditions are adverse for the durability (say sulphates or acidic soil fill). Such protection may be needed to the foundations, pedestals, columns or shaft up to about 0.6 m above ground, the ground or plinth brace. Protection may be necessary if the construction is on marshy land, creek, soil with high salinity or sulphates, etc.

17.4 For construction of column or wall sometimes 'kicker' (or starter) is specified, which may have a concrete height of 75 to 150 mm. Kicker is thought to facilitate construction by fixing formwork such that it cannot move, and reducing the slurry leakage from bottom end of formwork. Formation of kicker is an additional step in construction process.

NOTE – Provision of kicker nearly adds a construction joint. Kicker requires very small quantity of concrete, which is not properly attended, and the quality of concrete is usually poor. Hence, for good quality work kicker is not accepted. However, measures are necessary to fix formwork such that its bottom end is also prevented from movements, as well the slurry should not leak from the bottom joint.

18 APPURTENANCES AND MISCELLANEOUS ITEMS

Elevated liquid storage tank shall have following facilities and appurtenances.

18.1 Access to top of tank and inside tank shall be provided. Normally access is provided in three parts. First part is from ground level to the floor level of container or the bottom of wall of container. Second part of access is to roof of tank, and third part is from roof to floor of tank inside.

At or near the ground level, arrangement shall be such that un-authorized persons should not have access, for the safety of people, as well as safety of tank and threat of polluting the liquid, etc.

18.1.1 For the first two parts, steel ladder is commonly provided with landings at intermediate brace levels. Ladder shall be minimum 450 mm wide, the stringer may consist of $75\ \text{mm} \times 10\ \text{mm}$ MS flat (or $65\ \text{mm} \times 65\ \text{mm} \times 6\ \text{mm MS angle}$), steps of $2 \times 20\ \varphi$ deformed bars at 300 mm c/c. Ladder may consist of other materials and have different design.

18.1.2 For medium and large tanks (say $> 400\ \text{m}^3$) usually spiral stair case in RC is provided, utilizing precast RC steps. Such spiral stair is supported on a single core column, and braced to the staging. The minimum radius of step should be 1.2 m.

It is preferable to provide spiral stair at the centre of staging, to avoid eccentricity in dynamic behaviour. However, in such cases a cylindrical wall at centre of container is required for access to roof of container.

For large tanks alternately an independent staircase tower of four columns, having dog-legged stair may be provided. Stair should have railing or parapet. This type of stair tower contributes to eccentricity in dynamic behaviour of tank if connected to the main staging.

18.1.3 For going inside the tank, commonly ladder is provided. Steel ladder with anti-corrosive coating is the common material. Alternately, RC ladder or stair may be provided.

18.2 The elevated tank shall be provided with lightning arrester at its top. Height of arrester above the roof shall be such that the whole area of tank (including roof) is covered within conical protection zone. To reduce height of arrestor multiple arrestors can be provided. For large tank in plan, arrestor consisting of horizontal wire installation may be provided to protect large area. The arrestors shall be connected by conductor to the earthling arrangement. The whole arrangement shall confirm to IS/IEC 62305 (Parts 1 to 4) and the electrical rules. It shall be tested for the resistance as per rules.

18.3 The vertical assembly of pipelines should be adequately supported laterally to the members of staging by enough ties.

18.4 The vertical assembly of pipe shall have a proper foundation, which in most cases may consist of a concrete or masonry pedestal.

18.5 For shaft type staging, a door should be installed with lock and key arrangement. Adequate number of openings for ventilation and day light should be provided.

For framed staging, the access for going to top shall be provided with lock and key arrangement to prevent unauthorized access.

18.6 A suitable mechanical or electrical arrangement for indicating the level of liquid top in tank should be installed. This arrangement should be such that it is convenient to read the level from a position at ground.

18.7 Obstruction lighting or air space warning light shall be installed where required as per rules of aviation authorities in specified area, or if the total height of elevated tank exceeds 30 m from ground.

19 QUALITY MANAGEMENT IN CONSTRUCTION

For the properties of the completed structure to be consistent with the requirements and the assumptions made during the planning and design, quality assurance and execution expectations for construction shall be considered and adequate quality assurance measures shall be taken.

19.1 Quality Assurance Plan

It shall define the tasks and responsibilities of all persons involved in the design and construction, adequate control, and checking procedures and the organization and filing of an adequate documentation of the construction process.

19.2 Construction Requirements

The design note shall include requirements for the following:

a) Concrete mix proportions to meet performance requirements, evaluation, and acceptance of concrete conforming to IS 456.
b) Concrete placing and curing;
c) Quality control, sampling, and testing;
d) Details of shoring and re-shoring sequences;
e) Details of critical temporary conditions; and
f) Constructability.

19.3 A quality assurance plan should be prepared, to verify that the construction conforms to the design requirements and performance criteria. It should include the following:

a) Construction stages at which inspection and testing are required, forms for recording inspections and testing, and the qualification of personnel performing such work;
b) Procedures for exercising control of the construction work, and the personnel exercising such control; and
c) Methods and frequency of reporting, and the audit of quality reports.

ANNEX A
(*Clause* 2)

LIST OF REFERRED INDIAN STANDARDS

IS No.	*Title*
IS 269 : 2015	Ordinary Portland cement – Specification (*sixth revision*)
IS 383 : 2016	Coarse and fine aggregate for concrete – Specification (*third revision*)
IS 455 : 2015	Portland slag cement – Specification (*fifth revision*)
IS 456 : 2000	Plain and reinforced concrete – Code of practice (*fourth revision*)
IS 800 : 2007	General construction in steel – Code of practice (*third revision*)
IS 875	Code of practice for design loads (other than earthquake) for buildings and structures
(Part 1) : 1987	Dead loads Unit weights of building materials and stored materials (*second revision*)
(Part 2) : 1987	Imposed loads (*second revision*)
(Part 3) : 2015	Wind loads (*third revision*)
(Part 4) : 2021	Snow loads (*third revision*)
(Part 5) : 1987	Special loads and combinations (*second revision*)
IS 1080 : 1985	Code of practice for design and construction of shallow foundations in soils (other than raft, ring and shell) (*second revision*)
IS 1343 : 2012	Prestressed concrete – Code of practice (*second revision*)
IS 1489 (Part 1) : 2015	Portland pozzolana cement – Specification: Part 1 Fly ash based (*fourth revision*)
IS 1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement– Specifications(*fourth revision*)
IS 1893	Criteria for earthquake resistant design of structures
(Part 1) : 2016	General provisions and buildings (*sixth revision*)
(Part 2) : 2014	Liquid retaining tanks (*fifth revision*)
(Part 4) : 2015	Industrial structures including stack - like structures (*first revision*)
IS 1904 : 2021	General requirements for design and construction of foundations in soils – Code of practice (*fourth revision*)

IS No.	*Title*
IS 2911	Design and construction of pile foundations – Code of practice:
(Part 1/Sec 1) : 2010	Concrete piles, Section 1 Driven cast *In-situ* concrete piles (*second revision*)
(Part 1/Sec 2) : 2010	Concrete piles, Section 2 Bored cast *In-situ* concrete piles (*second revision*)
(Part 1/Sec 3) : 2010	Concrete piles, Section 3 Driven precast concrete piles (*second revision*)
(Part 1/Sec 4) : 2010	Concrete piles, Section 4 Precast concrete piles in prebored holes (*first revision*)
(Part 2) : 2021	Timber piles (*second revision*)
(Part 3) : 2021	Under-reamed piles (*second revision*)
(Part 4) : 2013	Load test on piles (*second revision*)
IS 2950 (Part 1) : 1981	Code of practice for design and construction of raft foundations: Part 1 Design (*second revision*)
IS 3370	Concrete structures for retaining aqueous liquids – Code of practice:
(Part 1) : 2021	General requirements (*second revision*)
(Part 2) : 2021	Plan and reinforced concrete (*second revision*)
(Part 3) : 2021	Prestressed concrete (*first revision*)
IS 3812 (Part 1) : 2013	Pulverized fuel ash – Specification: Part 1 For use as pozzolana in cement, cement mortar and concrete (*third revision*)
IS 4926 : 2003	Ready-mixed concrete – Code of practice (*second revision*)
IS 4991 : 1968	Criteria for blast resistant design of structures for explosions above ground
IS 6922 : 1973	Criteria for safety and design of structures subjected to underground blasts
IS 9103 : 1999	Concrete admixtures – Specification (*first revision*)
IS 9456 : 1980	Code of practice for design and construction of conical and hyperbolic paraboloidal types of shell foundations
IS 11089 : 1984	Code of practice for design and construction of ring foundation
IS 13620 : 1993	Fusion bonded epoxy coated reinforcing bars - Specification
IS 13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces – Code of practice (*first revision*)
IS 14687 : 1999	Falsework for concrete structures – Guidelines
IS 16354 : 2015	Metakaolin for use in cement, cement mortar and concrete – Specification
IS 16481 : 2022	Textiles - synthetic micro fibres for use in cement based matrix – Specification (*first revision*)
IS 16651 : 2017	High strength deformed stainless steel bars and wires for concrete reinforcement – Specification
IS 16714 : 2018	Ground granulated blast furnace slag for use in cement, mortar and concrete – Specification

IS No.	*Title*
IS/IEC 62305	Protection against lightning:
(Part 1) : 2010	General principles
(Part 2) : 2010	Risk management
(Part 3) : 2010	Physical damage to structures and life hazard
(Part 4) : 2010	Electrical and electronic systems within structures

ANNEX B
(*Clause* 8.6)

TYPES OF STAGING

B-1 Elevated tanks can be classified based on the liquid capacities in m^3.

a)	Very small	up to 50 m^3,
b)	Small	51 to 150 m^3,
c)	Medium	151 to 500 m^3,
d)	Large	501 to 1 500 m^3,
e)	Very large	above 1 500m^3.

B-2 Elevated tanks in plan can be circular, square or rectangular.

B-2.1 Square and rectangular tanks are planned on square grid, may be some times on rectangular grid of columns. These shapes are adopted when owner's requirement or availability of land restricts the shape. Square tanks may be on 4, 9, 16, 25, 36, etc columns.

Except for small tank (< 30 m^3), walls of rectangular tanks are normally costlier than the circular wall. For a given area of tank, circular wall requires least perimeter and formwork.

B-2.2 Circular tanks can be planned on different configurations of column layout.

B-2.2.1 Columns on square grid.

These tanks have cylindrical wall. The floor and roof of tank consist of slab beam system. Slab-beams of roof may be replaced by flat-slab system. The possible number of columns can be 4, 9, 12, 16, 21, 24, 32, 37, 44, 52, etc for a tank. Figures 2, 4, 5, 25, 26 and 27 show the typical configurations at floor plan for cylindrical containers. These configurations are common due to overall economy.

NOTE – Flat-slab system (beam less) is normally not recommended for floor of a tank. If it is used, the analysis should be done by finite element method (and not by direct method as per **31.4**, IS 456), and high ductility should be imparted to column strip for its behaviour as part of moment resisting frame.

B-2.2.2 Of these, more preferred and convenient for construction are the configurations with 4, 12 and 24 columns typically shown in Figs 2, 4, 5, Fig 25 (a), Fig 25(b), Fig 25(e) and Fig 25(i).

Spacing of columns is governed by span of slab which influences the economy. The spacing ranges from 3 to 5 m and increases with bearing pressure of foundation and also with height of staging.

By using secondary beams supported on primary beams spanning over columns, the span of slab can be reduced to half or one third. *see* typical examples in Figs. 4, 5 and 26. Thus with secondary beams, spacing of column may range from 4 m to 8 m.

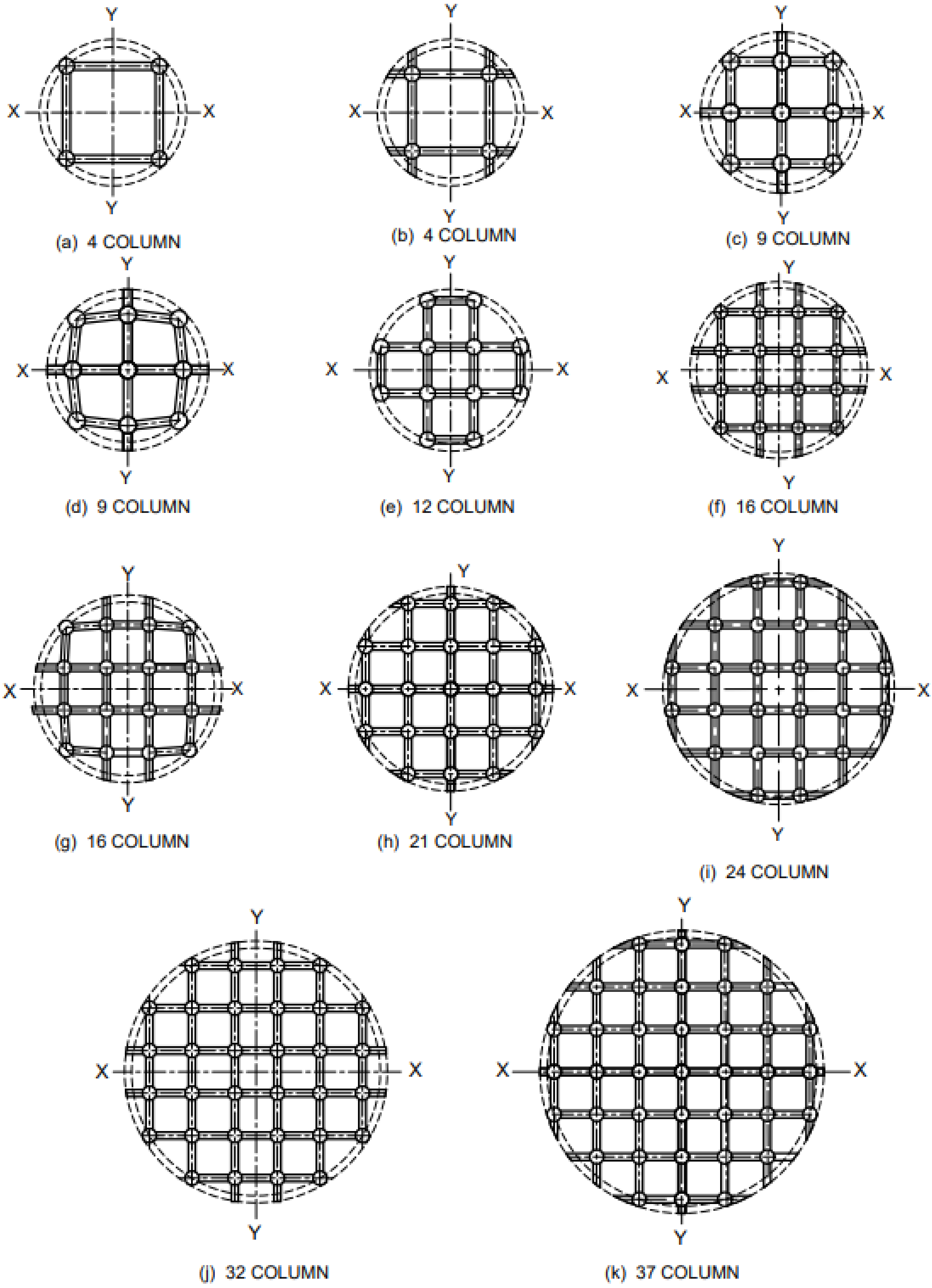
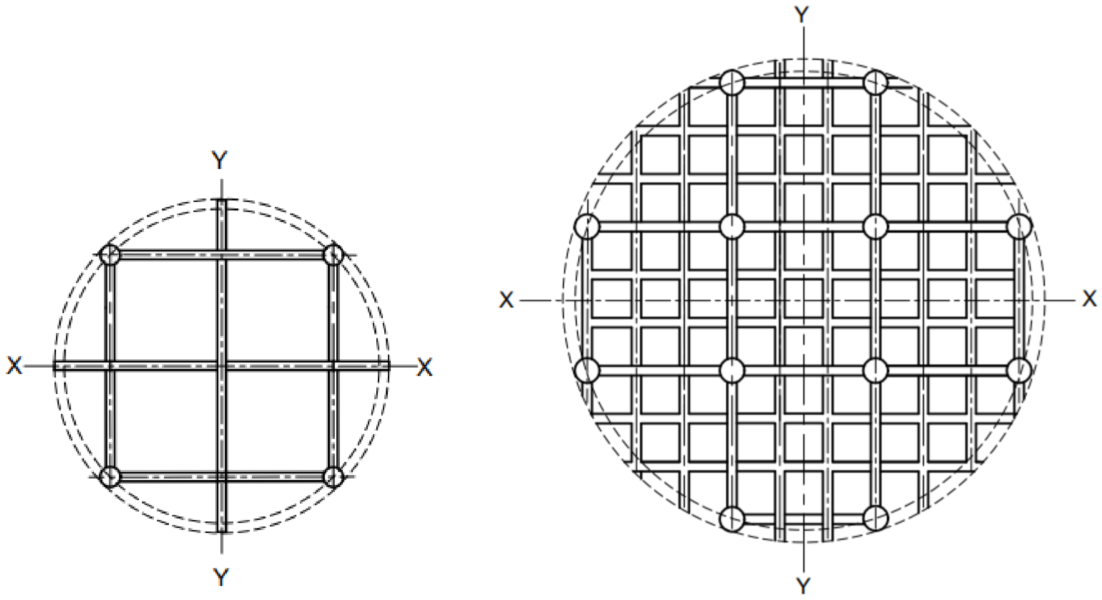


FIG. 25 FLOOR PLANS OF CYLINDRICAL TANKS.



26 (a) 26 (b)

FIG. 26 FLOOR PLANS OF CYLINDRICAL TANKS WITH SECONDARY BEAM.

B-2.2.3 Circular tanks on square grid, has most slab panels square. Slab panels having arc of circle and nearly triangular shapes require higher cost of formwork as standard shuttering plates are not useful. Configurations having higher percentage of slab area requiring non-square panels increase the cost of tank. Shuttering material in odd shape requires cutting it to pieces and reduces their reuse, making it costly. For such portions, tendency is to cut cost by use of inappropriate material affecting the quality greatly. Simplicity in construction can also give improved quality and cost reduction which are desirable.

B-2.2.4 Cylindrical tanks up to 500 m³ have been constructed on 4 column staging, and large tanks on 24 columns. Staging with lesser numbers of columns are generally economical in construction and appears aesthetically better, though consumes little more concrete and steel (may be 4 to 8 percent). For medium tanks also four column staging produces efficient and stable design, and are easy to construct. It is not advantageous to insist on more number of columns for small and medium tanks.

B-2.2.5 In configurations of 12, 16, 24, 32, 44, 52 etc columns, it is possible to have number of braces at a level to be equal to number of columns that is, for 24 column staging brace at a level can also be 24. Typical examples of plans of these staging are shown in Fig 27. The braces are not continuous over all the spans, but almost on alternate spans. However, the ground level braces should be continuous. This configuration makes the construction economical and fast. However, the dynamic behaviour of the staging changes in modal analysis. Hence, a little lower value of response reduction factor is recommended for such configurations.

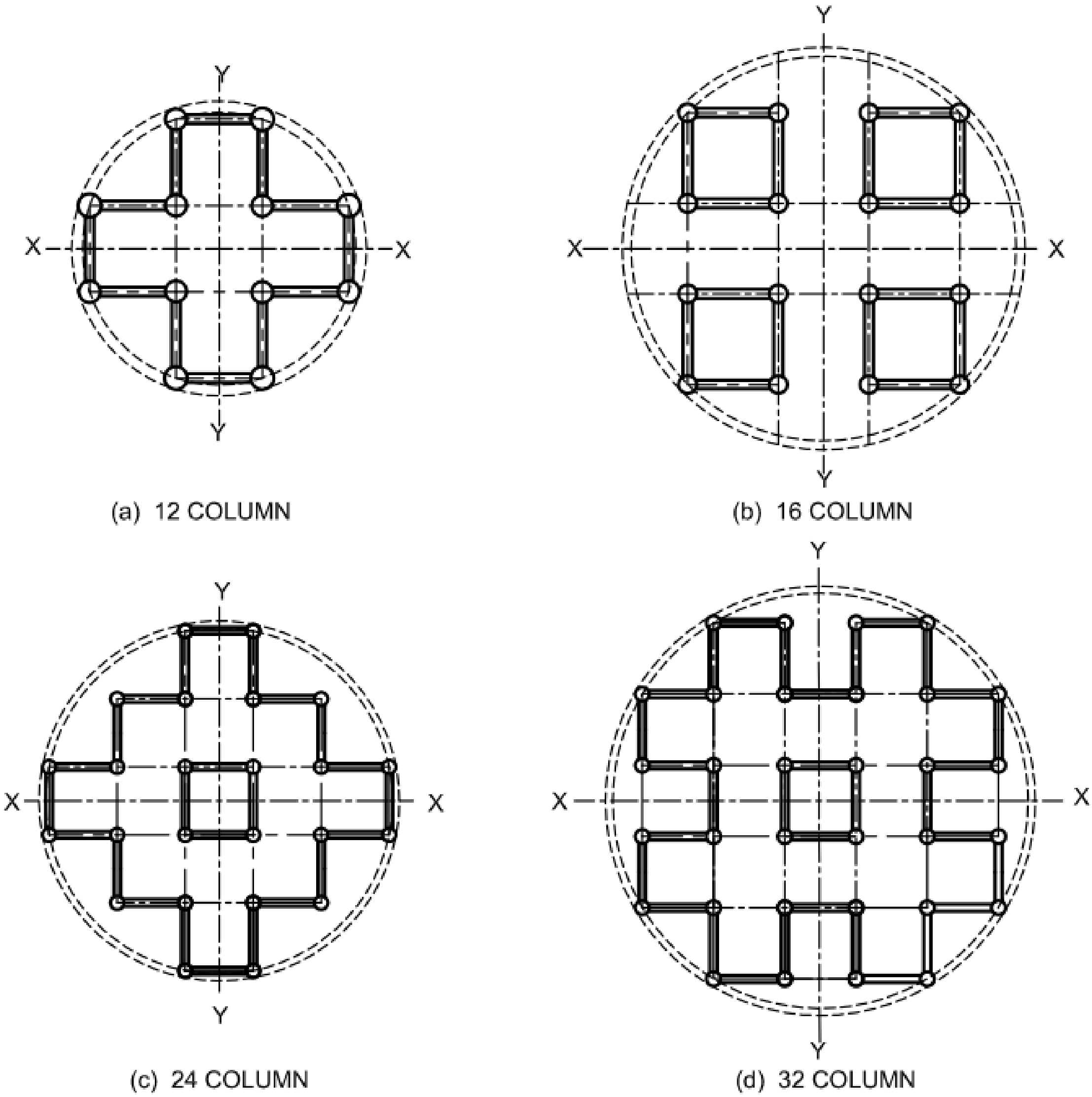


FIG. 27 PLANS OF STAGING WITH BRACE IN ALTERNATE BAY.

B-2.2.6 Circular tank on five columns (*see* Fig 28), of which one is at centre of staging, have been constructed to reduce the span of slab. However, this configuration makes construction costly as total number and length of braces and floor beams are very high, as well, slab is divided in to triangular panels.

B-2.2.7 For a tank of any shape generally less numbers of columns, gives it an better look.

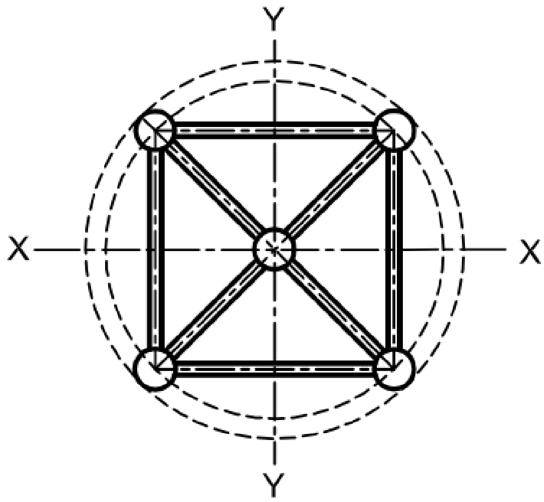


FIG. 28 PLAN OF STAGING WITH 5 COLUMN.

B-2.3 Cylindrical tanks may also be on columns placed on radial and circumferential grid pattern (*see* Fig. 29). In this pattern, the columns are placed on circles two or more at different radius. Columns are connected by braces, which are along radial and circumferential direction. The floor of the tank can consist of slab and beam layout similar to braces. In central portion (over the smallest circle), floor slab or floor dome may be provided. The layout of roof is usually same as that of floor.

These types of tanks may be provided for elevated units of treatment plants. These shapes are not common now for elevated tanks.

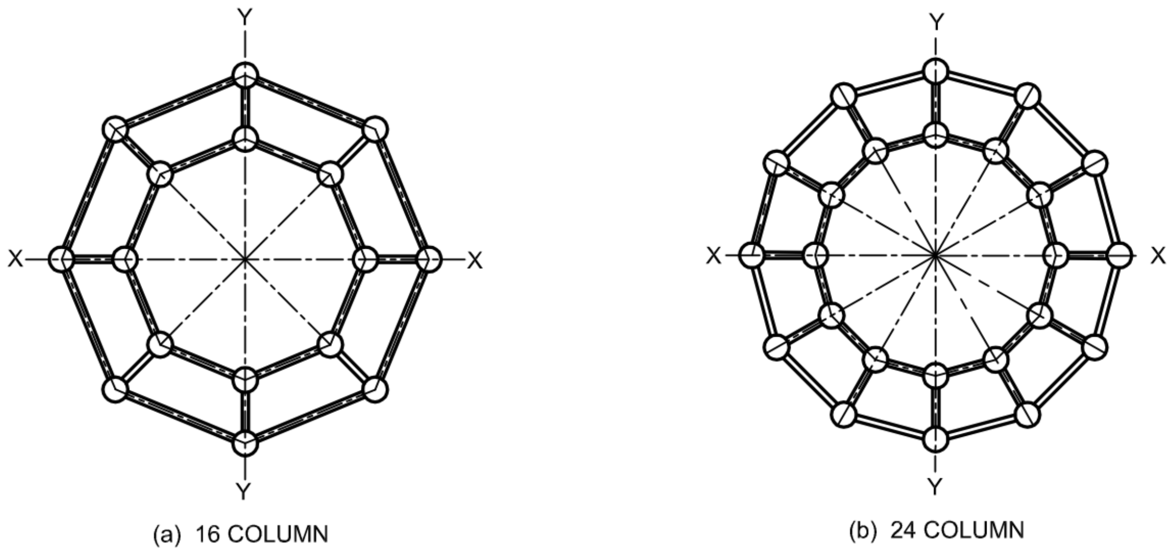


FIG. 29 PLANS OF STAGING WITH COLUMNS ON RADIAL AND CIRCUMFERENTAIL GRID.

B-2.4 Circular staging may have all columns on a circle only. Typical examples are given in Fig. 30 as plans of staging showing brace arrangements. The tank shape may be Intze or cylindrical with domed bottom. Very small (or small) tank may have slab bottom in place of dome. These tanks can also be constructed on shaft (cylindrical shell) type staging. However, up to medium capacity, tanks are not economical with shaft staging.

a) Domed bottom cylindrical tanks can be constructed for small and medium capacities. These are difficult to construct and usually costlier than the slab bottom cylindrical tank. The formwork for the dome consumes lot of cost and time of construction and the liquid body has larger surface area.
b) Compared to 4 columns, with slab beam bottom, small cylindrical tanks (100 m^3 to 450 m^3) on 5 or 6 columns poses more difficult construction, and does not turn out to be economical. Ease in construction with 4 column tank, also gives quality and economy.

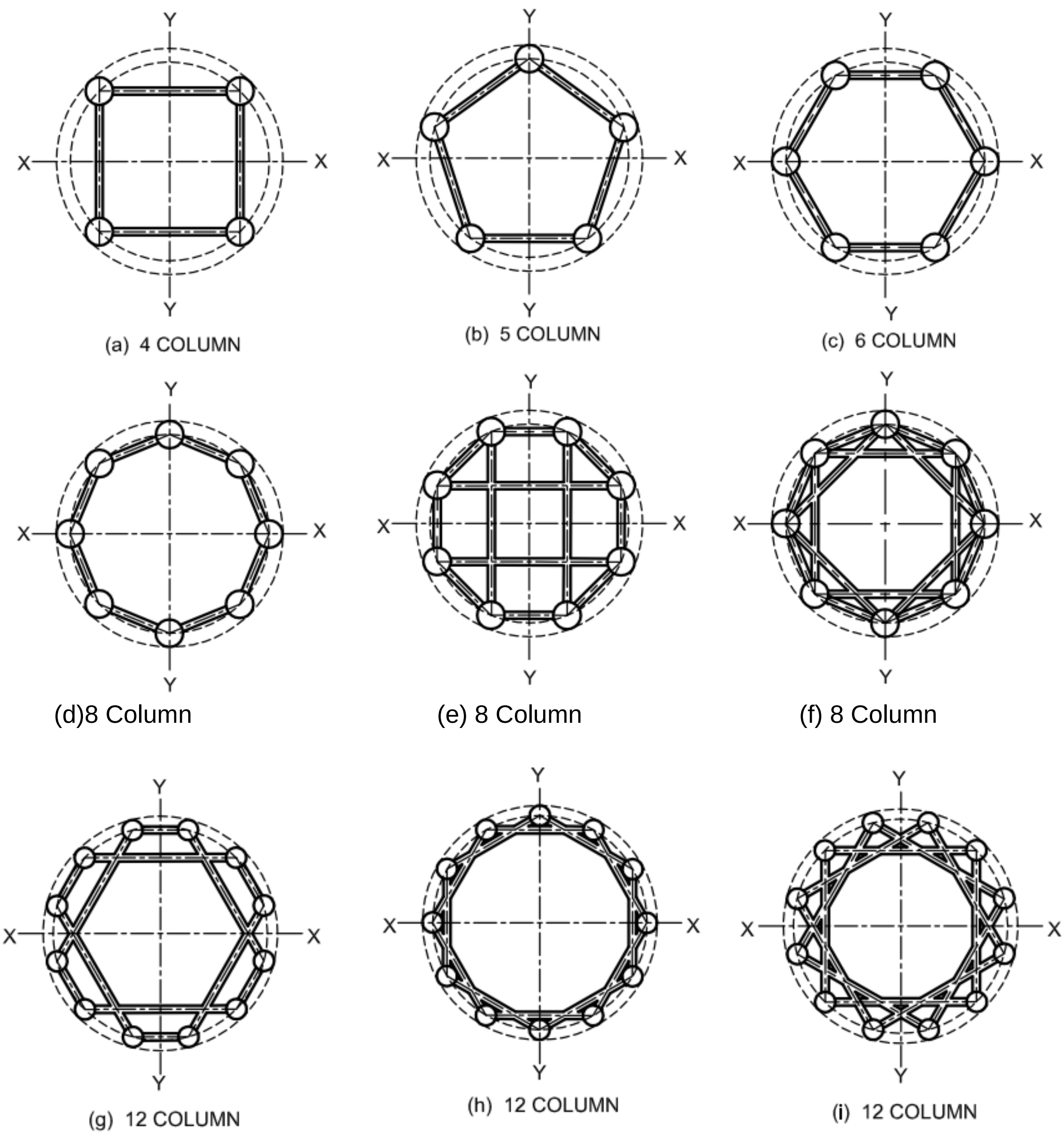


FIG. 30 PLANS OF STAGING WITH COLUMNS ON A CIRCLE.

B-2.5 Depth of liquid in tank is governed by hydraulic considerations. The energy demand during service life requires a smaller depth of liquid for tanks, which may be 3 m for small tanks and up to 5 m for large tanks. However, large tanks can be constructed with depth of liquid up to 8 m. The cost of energy difference over the life is usually many times more than the savings in capital cost of tank by increasing the liquid depth (from life cycle cost consideration). Larger depth may be required where land availability is restrictive.

B-3 RECOMMENDATIONS FOR RANGE OF TANK CAPACITIES

B-3.1 Tanks of capacity up to 30 m^3 may be square in plan and supported on four columns. These may be cylindrical also.

B-3.2 For tank up to 150 m^3 staging with four columns is suitable. Intze tanks up to 500 m^3 have been constructed on four columns. Cylindrical tanks with floor slab and beams having cantilever, have been constructed up to 500 m^3 capacity, though method is suitable for tanks up to 300 m^3. Adding a column at centre of staging, and other columns on a circle require more number of floor beams and brace, thus such layout is not economical.

B-3.3 For medium to large capacity (151 m^3 to 1 500 m^3) the tank may be circular, Intze type, or square/rectangular. The number of columns adopted shall be decided based on the column spacing which normally lies between 3 m and 4 m for economy. Spacing of columns can be up to 6 m if secondary beam divides the slab span to half, and up to 8 m where two secondary beams divide the span to a third. For circular, Intze or conical tanks, a shaft type staging can be provided for tanks above 500 m^3.

Conical tank on shaft staging is quite a costlier form, but usually chosen for its aesthetic appeal. Shaft staging may not be economical for small and medium capacity range.

For medium and up to 1 000 m^3 capacities, it is more common to have cylindrical tanks on 12 columns on square grid, more especially if liquid depth permitted in tank is small (say about 3 m); and floor consisting of slab-beam system. In general staging of cylindrical tank with columns (say six or more in numbers) on single circle are not suitable for the capacity above 500 m^3.

B-3.4 Large tanks (> 500 m^3) can be cylindrical or Intze shape. Cylindrical tank can be designed for a depth of liquid of 3 m to 5 m usually with beam column floor on square grid. Intze tank will require 5 m to 8 m depth of liquid. Either if depth of liquid is low or if allowable bearing pressure of founding soil is low, cylindrical tank with columns on square grid, gives substantial economy. In most cases cylindrical tank with floor slab and beam are easier to construct and gives an economical solution compared to Intze tank Compared to cylindrical or rectangular tanks, Intze shape is aesthetically appealing.

B-4 INTZE TANKS

B-4.1 Staging with 12 or more columns on a circle is very sensitive to construction error (column out of plumb or radial distance varying), and also for critical buckling load, because the angle between braces at column is very high (say $> 150°$) if internal braces (that is, other than circumferential braces) are not provided. To reduce the angle, additional braces called internal braces can be provided. Intze tanks are more costly if bearing capacity of foundation is low. Intze tanks are popular on shaft staging rather on columns.

B-4.2 Formworks for shells like dome and cone are complicated and highly skillful, requiring experienced expert workers, hence costly. These formwork has major wooden elements, without joinery details, which is highly prone to errors and accidents. Such formwork is not amenable to quality management system as every joint/connection needs number of rechecks, non-repetitive nature, and of low reliability.

Shells of revolution (for example, cone and dome) have sloping and curved surfaces. Concrete placement on the sloping surface (for cone or dome) cannot be compacted properly. To vibrate the concrete in cone and portion near springing of dome, top-form or back-form, and the hold down arrangement or ties to prevent top shutter from uplift are necessary. In most cases, these top forms are not provided and quality of the concrete may remain unreliable. Such concrete is not acceptable for water-tightness and durability. Due to very costly formwork and low reliability of quality concrete, Intze tanks are not preferred now-a-days. The formwork for shells of revolution like dome and cone for Intze shape and domed bottom are specialized, skilled job, non-repetitive, and very costly, which normally offsets the savings in the materials (concrete and steel), thus pushing up the overall cost of tank.

B-4.3 Intze shape or domed bottom tank require sizeable depth of liquid in tank.

B-4.4 For minimizing material cost in tank, smaller spacing of columns say 3 m to 5 m is apparently adopted. But for economy in overall construction, reduced number of members in staging, that is higher spacing of about 4 m to 6 m can be selected. Spacing greater than 6 m may not be ruled out. Optimising total length of columns, braces and beams gives economy.

B-5 Besides those discussed in **B-2** to **B-4**, unusual shapes such as spherical or multi-cell may be adopted as per discretion of the designer and aesthetic requirements of the owner.

ANNEX C
(*Clause*8.6)

STRUCTURAL CONFIGURATION OF MEMBERS

C-1 Generally the shape of tank, configuration of members of tank and staging, span of main members, and layout of staging for economical design, will be governed by the functional requirements some of which are enumerated below.

a) Capacity of tank (Q_c in m³);
b) Height of staging;
c) Maximum liquid depth (h) allowable by functional / hydraulic design;
d) Net safe bearing pressure (SB) of foundation strata and type of foundation suitable; and
e) Other site conditions.

C-1.1 Average capacity supported by a column, that is ratio of tank capacity (Q_c) to number (N_c) of column = Q_c / N_c, increases with capacity of tank, and also with height of staging.

C-1.2 As the allowable bearing pressure of foundation strata reduces (below 200 kN/m²), Q_c / N_c will also reduce for economy.

C-1.3 The total cost of container and staging reduces, as liquid depth in the tank can increase, if allowable bearing pressure of foundation permits the increase in depth (*see* **C-1.5**).

C-1.4 If inflow to the tank is by pumping, the operational cost reduces with liquid depth due to saving in energy. In such cases increase in recurring energy cost over the design life will be very much higher than saving in cost of tank by increasing the depth of liquid.

Where inflow to the tank is by gravity, the maximum head available can be utilised for keeping depth of liquid more to economize on tank cost.

C-1.5 For low allowable bearing pressure of founding soil (BC), the liquid depth (H) in tank has to be limited. Relation of liquid depth to bearing capacity can be as below.

$$(\mathrm{H} \times \text{liquid density}) \leq \mathrm{BC} / K_{\mathrm{bh}} \text{ or } \mathrm{H} \leq \mathrm{BC} / (K_{\mathrm{bh}} \times \text{liquid density})$$

where, H = depth in m, & BC = allowable bearing pressure in kN/m².

The ratio K_{bh} may not be less than 18 to 20 for full mat or raft foundation. For economical design (that is, for low ratio of concrete volume to tank capacity), it could be 18. It increases with staging height and also increases with the thickness of members. As the ratio of *DL* (tank + staging) to *FL* in tank increases, K_{bh} shall increase. The ratio of loads is usually 0.7 to 1.0.

For low allowable bearing pressure, mat foundation is required covering the whole area of tank. Here, bearing capacity governs the design depth of liquid in tank.

Strip foundation or individual footings can be provided, if the ratio K_{bh} is significantly larger than the recommended value (that is, > 22). Individual footings are possible only if K_{bh} is substantially higher than the range recommended (that is, greater than 25).

C-2 Reduction of total surface area of elements of staging, reduces the wind load on staging, which otherwise for higher staging heights (say > 20m) increases the cost. Reduction of surface area of staging also amounts to reduction of formwork area for staging.

Construction aspects influence the cost of elevated tank significantly. Less number of columns or larger spans, though increases the quantity of steel and concrete slightly, however may reduce the cost of working due to reduction in formwork item (in m^2). Reduction of total length of column and braces can be done by reducing the number of columns (increase Q_{c} / N_{c}) and also reducing number of braces to derive economy in total construction cost. Significant part of the working cost (other than material cost of concrete and steel) depends on the total length of column and braces.

C-3 Slab-beam systems are common, and with cylindrical wall can give significant cost reduction. Domes and cones are specialized works, and formworks are costly. Also, due to complexity of the work, the quality and reliability could be low.