AN INTRODUCTION TO
WELDED TANKS FOR OIL STORAGE, API STANDARD 650

(TWELFTH EDITION, JANUARY 2016)

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CONTENT

CONTENT

BASIC INFORMATION

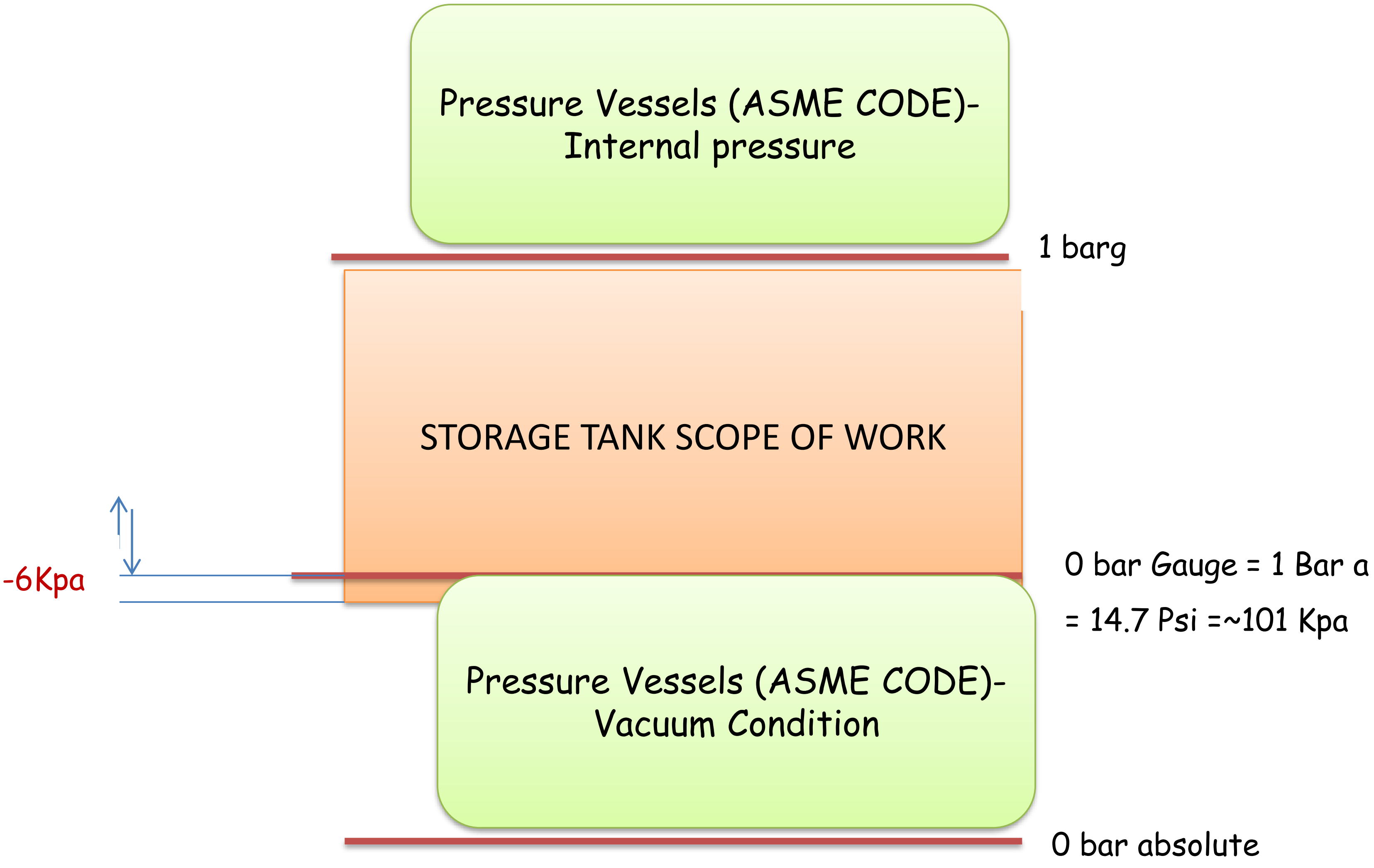
SCOPE

- BOTTOM IS UNIFORMLY SUPPORTED
- TANKS IN NON-REFRIGERATED SERVICE
- MAXIMUM DESIGN TEMPERATURE OF 93 °C
- MAXIMUM DESIGN INTERNAL PRESSURES OF 18 KPA
- MAXIMUM DESIGN EXTERNAL PRESSURE OF 6.9 KPA

NOTE:

A bullet (•) at the beginning of a paragraph indicates that there is an expressed decision or action required of the Purchaser. The Purchaser's responsibility is not limited to these decisions or actions alone. When such decisions and actions are taken, they are to be specified in documents such as requisitions, change orders, data sheets, and drawings.

STANDARD INTRODUCTION



Pressure Vessels (ASME CODE)- Internal pressure

1 barg

STORAGE TANK SCOPE OF WORK

-6Kpa

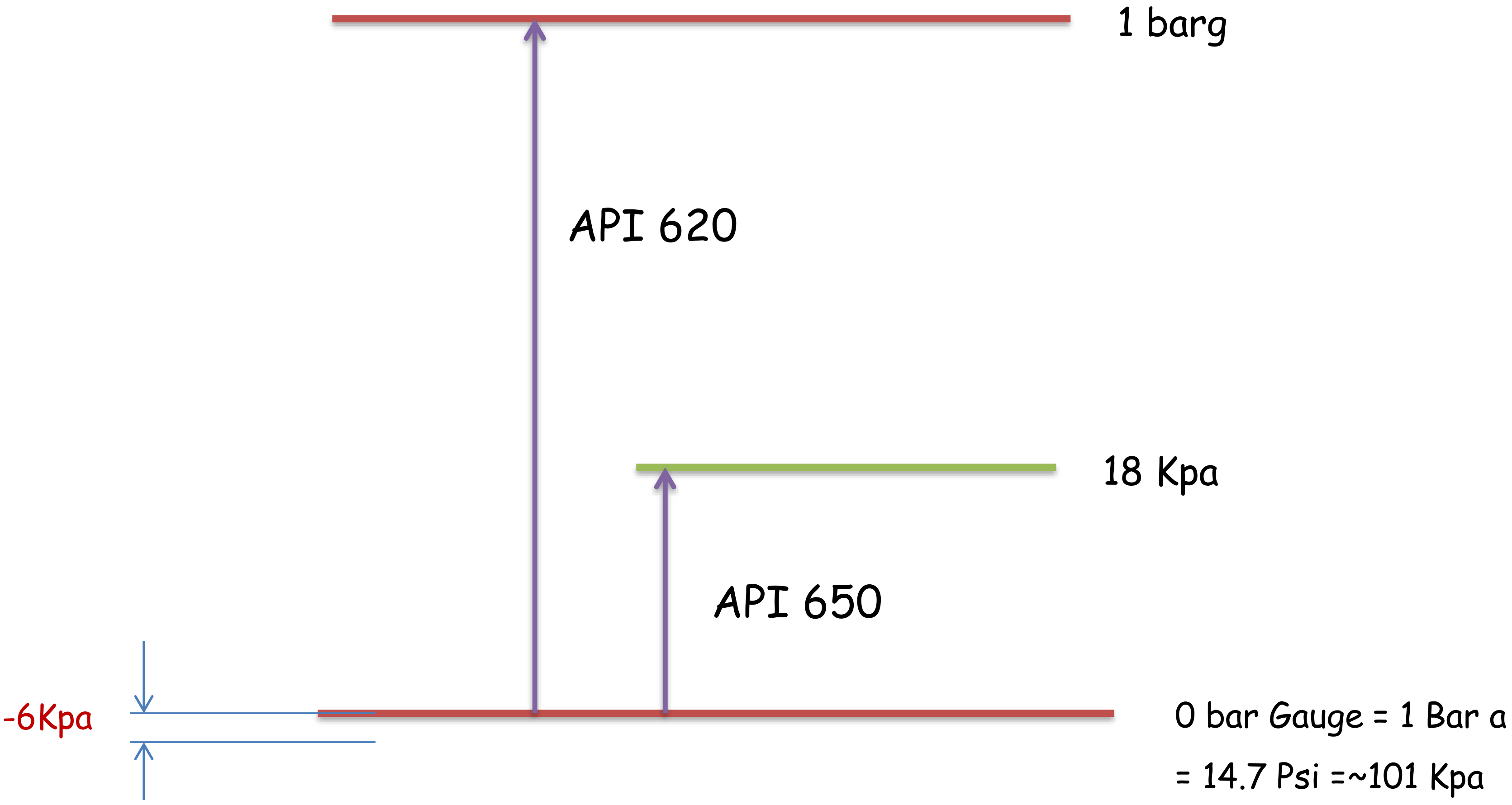
0 bar Gauge = 1 Bar a

= 14.7 Psi =~101 Kpa

Pressure Vessels (ASME CODE)- Vacuum Condition

0 bar absolute

STANDARD INTRODUCTION

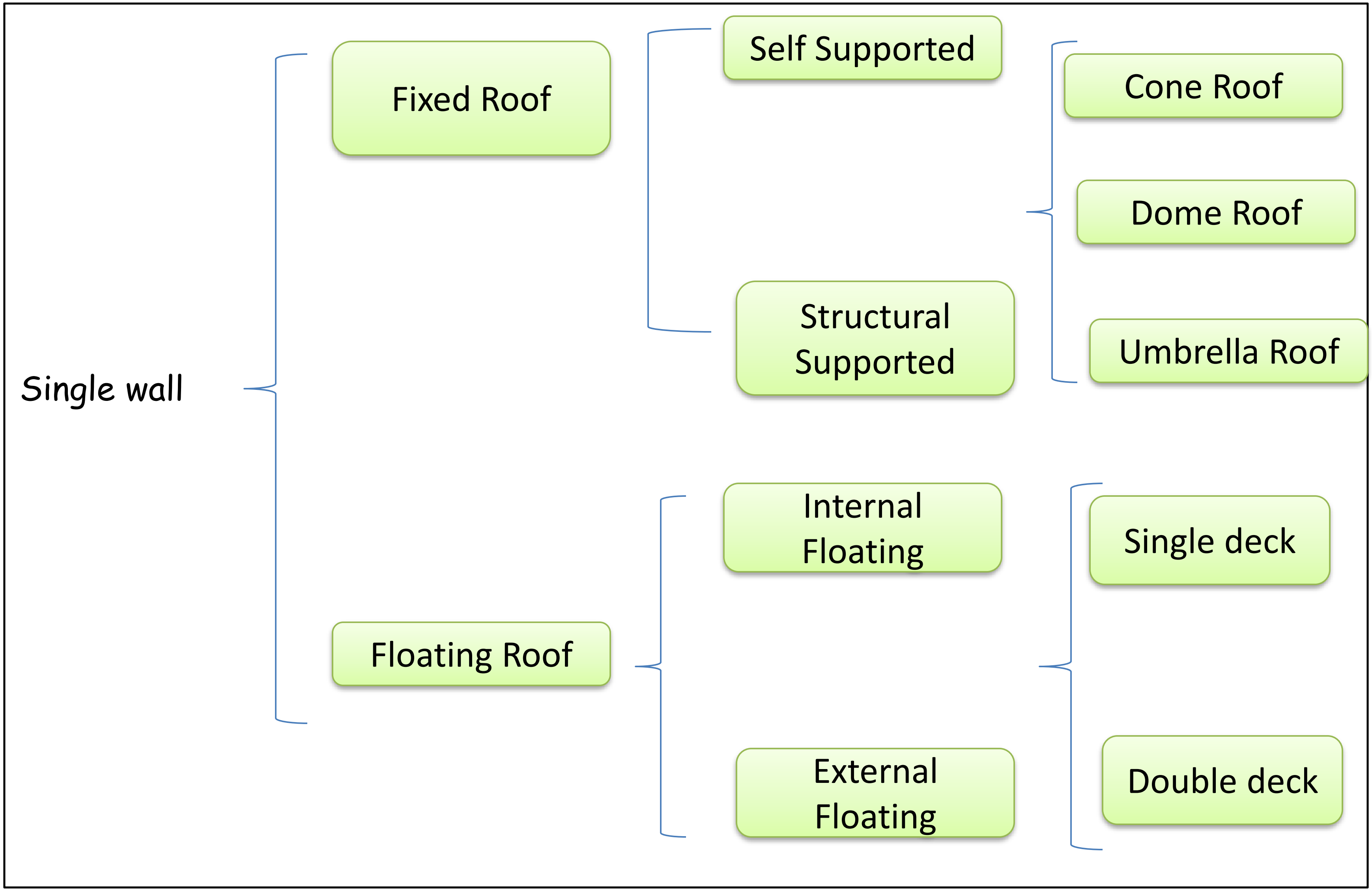


Type of Tanks

Type of tanks

- Single wall
- Double wall
- Elevated tank
- Buried tank
- Rectangular tank
- Spherical tank

Type of Tanks



Single wall

Fixed Roof

Self Supported

Structural Supported

Cone Roof

Dome Roof

Umbrella Roof

Floating Roof

Internal Floating

External Floating

Single deck

Double deck

STANDARD INTRODUCTION

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Section 3 : Terms and Definitions

Section 4 : Materials

Section 5 : Design

Section 6 : Fabrication

Section 7 : Erection

Section 8 : Methods of Examining Joints

Section 9 : Welding Procedure and Welder Qualifications

Section 10 : Marking

Annex A : Optional Design Basis for Small Tanks

Annex B : Recommendations for Design and Construction of Foundations for Aboveground Oil Storage Tanks

Annex C : External Floating Roofs

STANDARD INTRODUCTION

Annex D : Inquiries and Suggestions for Change

Annex E : Seismic Design of Storage Tanks

Annex F : Design of Tanks for Small Internal Pressures

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Annex H : Internal Floating Roofs

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Annex L : API Standard 650 Storage Tank Data Sheet

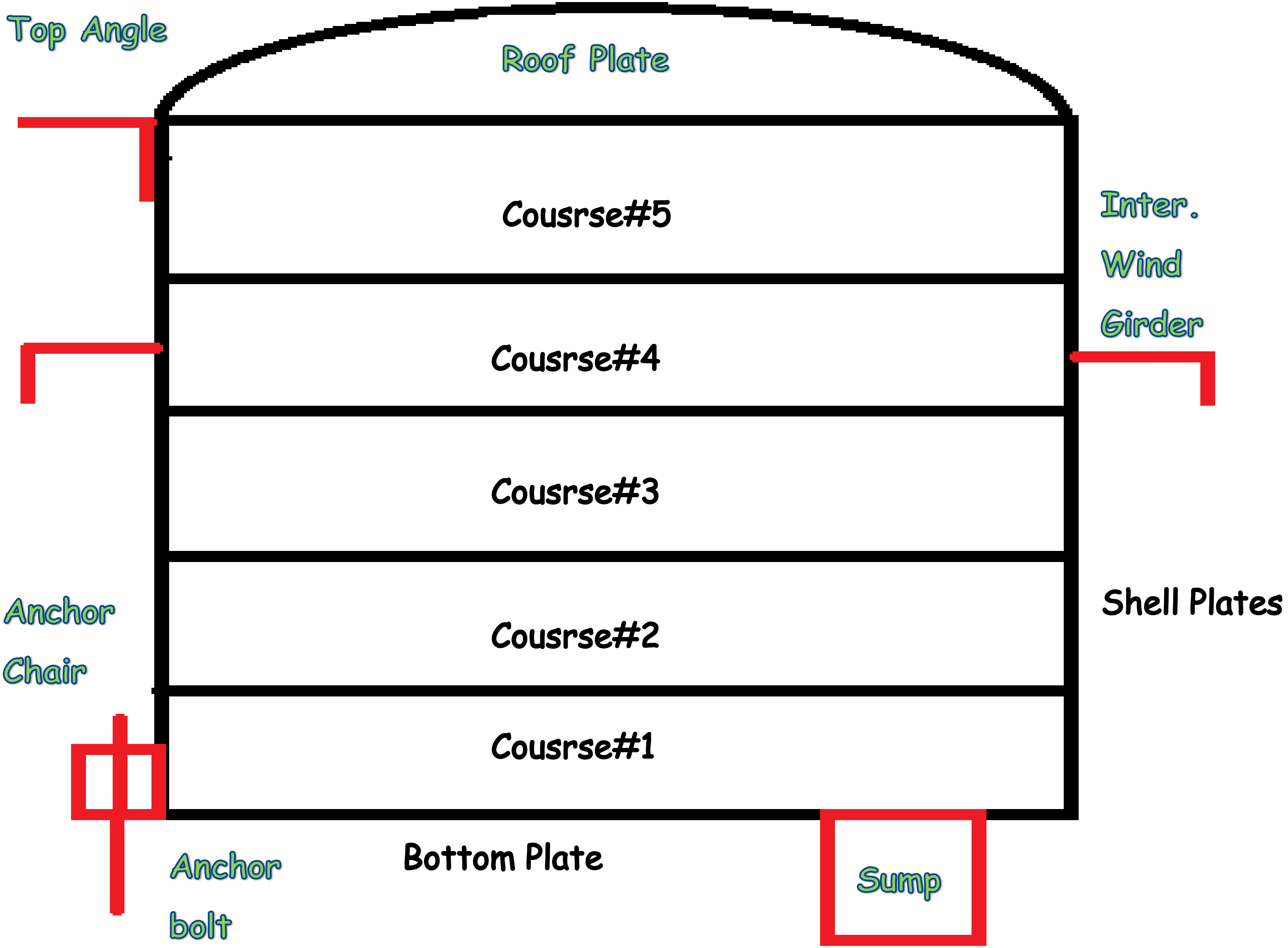
Annex M : Requirements for Tanks Operating at Elevated Temperatures

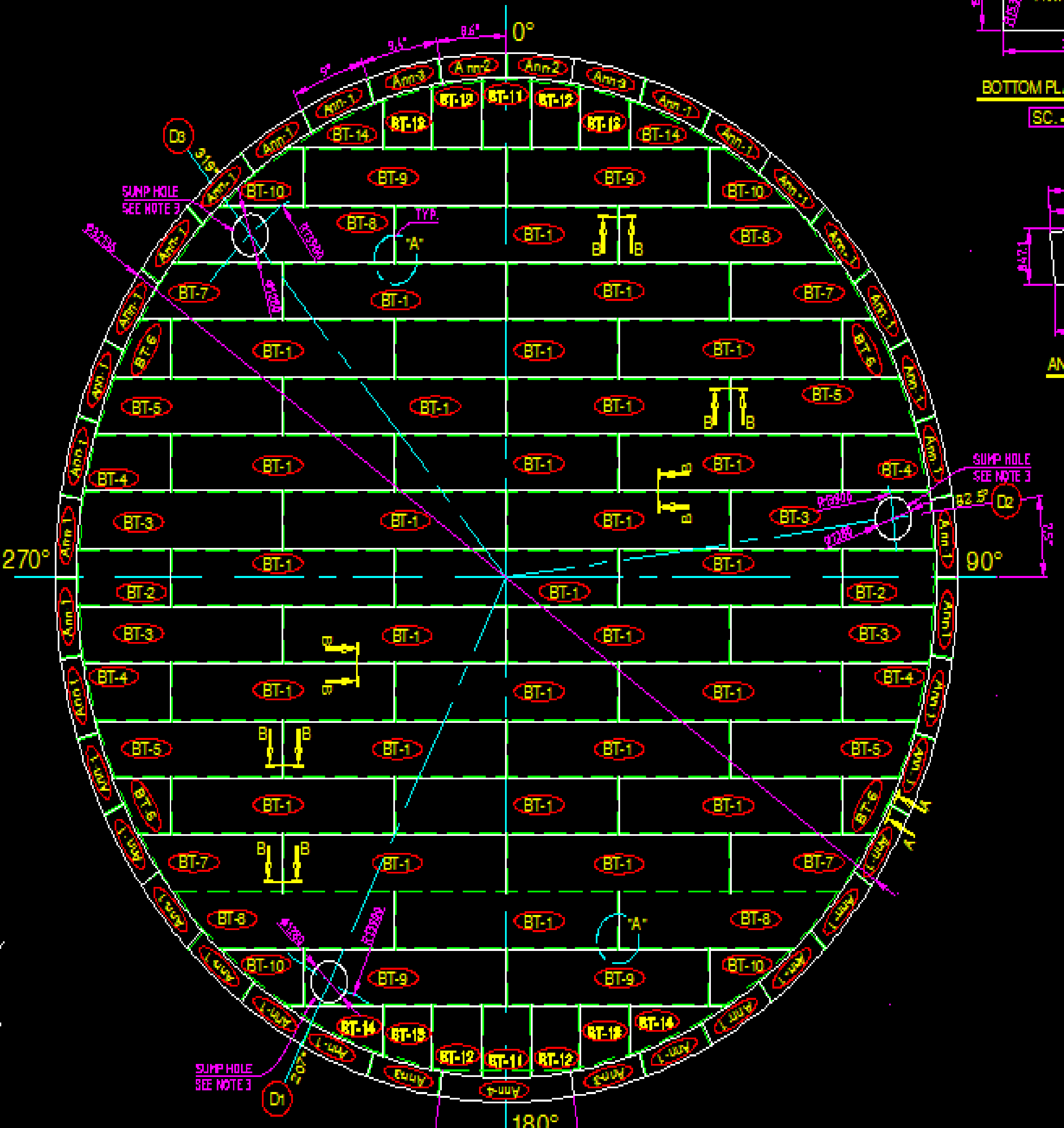
Annex P : Allowable External Loads on Tank Shell Openings

Annex S : Austenitic Stainless Steel Storage Tanks

Annex V : Design of Storage Tanks for External Pressure

STORAGE TANK PARTS INTRODUCTION


Top Angle
Roof Plate
Cousrse#5
Inter.
Wind
Girder
Cousrse#4
Cousrse#3
Anchor
Chair
Cousrse#2
Shell Plates
Cousrse#1
Anchor
bolt
Bottom Plate
Sump

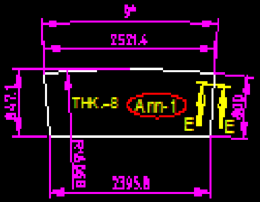


BOTTOM PLATE-14 (BT-14)

SC. = 1 :80

BOTTOM PLATE-5 (BT-5)

SC. = 1 :80



ANNULAR PLATE (Ann-1)

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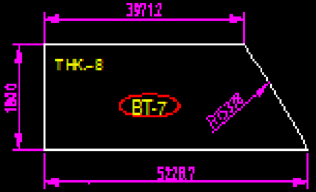
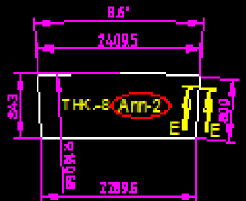


BOTTOM PLATE-7 (BT-7)

SC. = 1 :80



ANNULAR PLATE (Ann-2)

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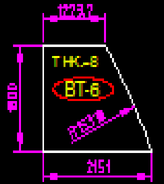

BOTTOM PLATE-6 (BT-6)

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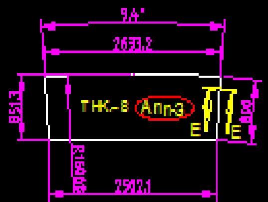

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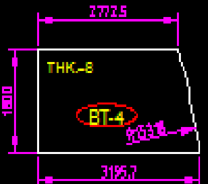

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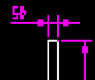
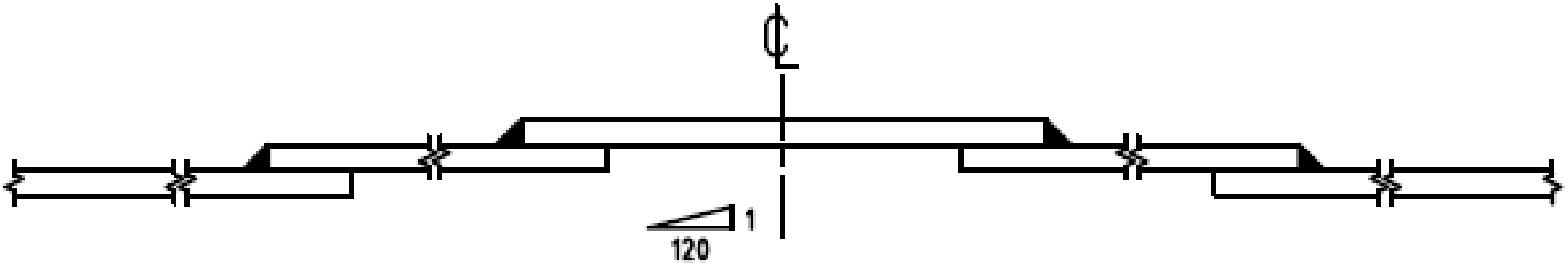
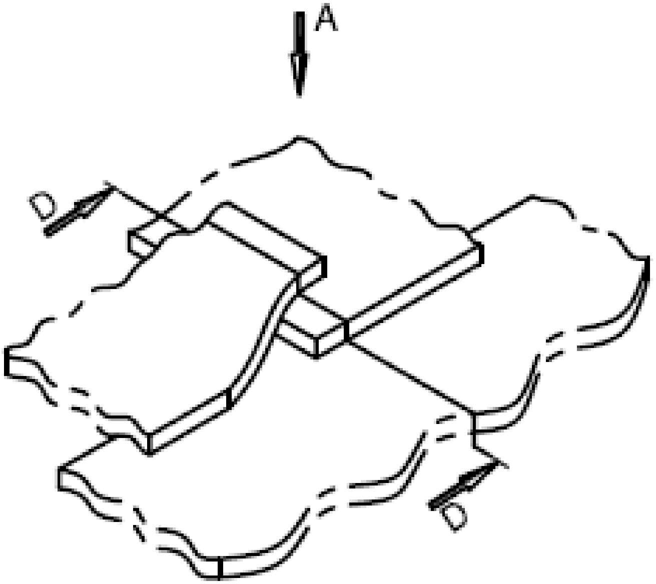
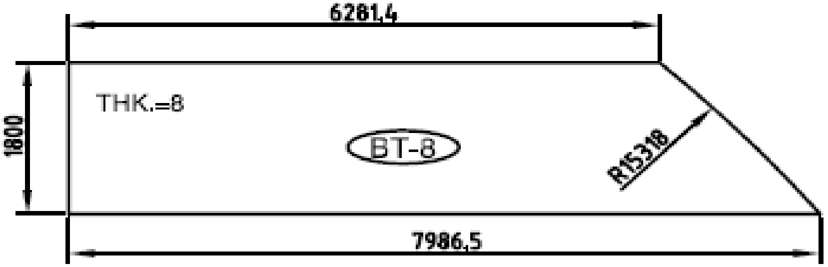




BOTTOM SLOPE





BOTTOM PLATE-8 (BT-8)

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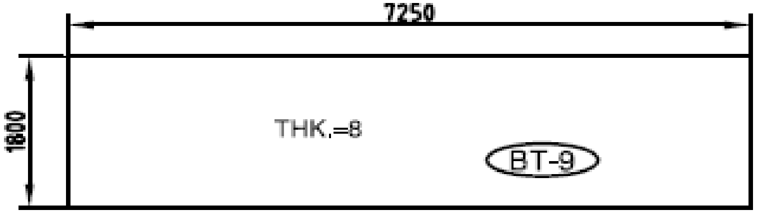
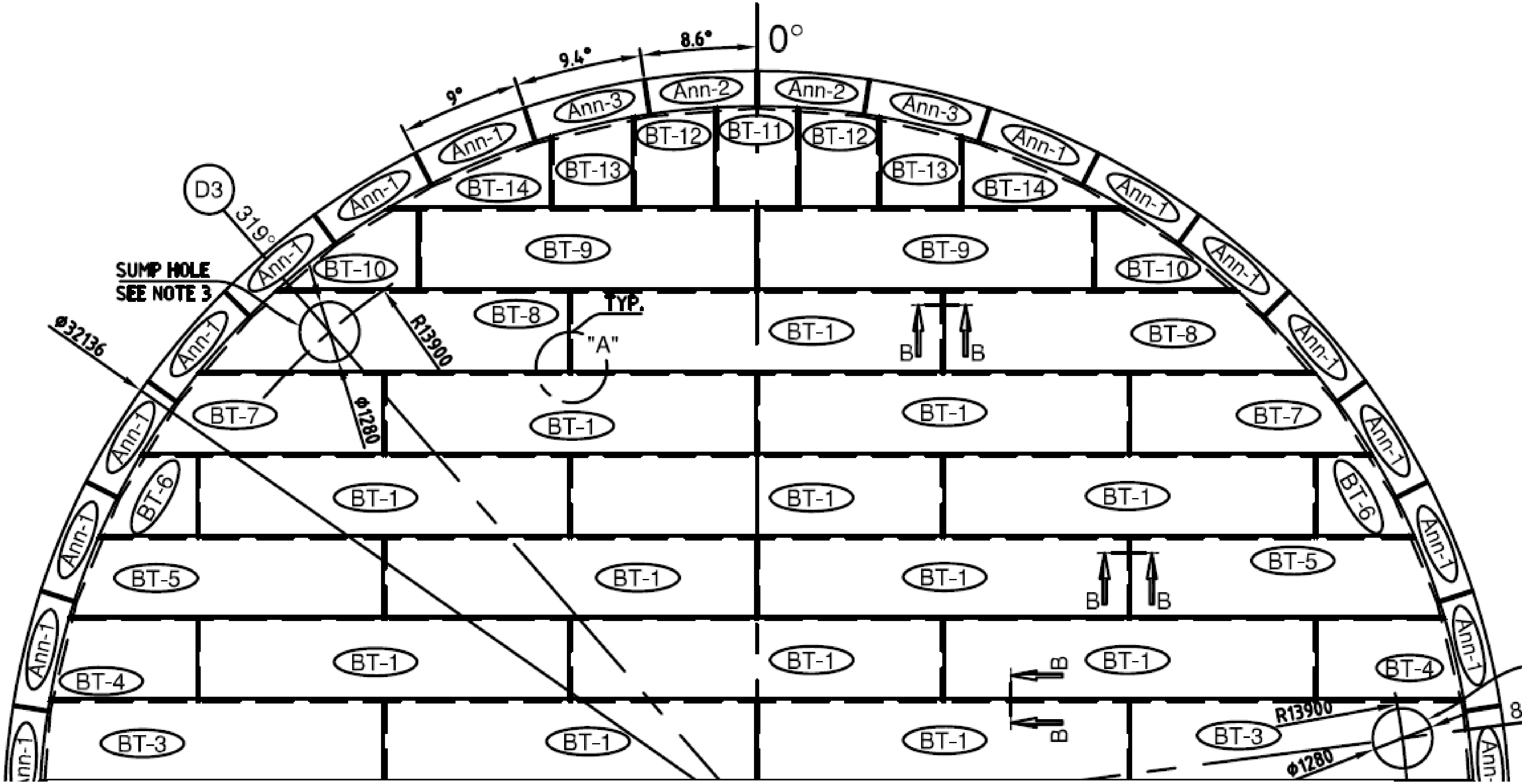
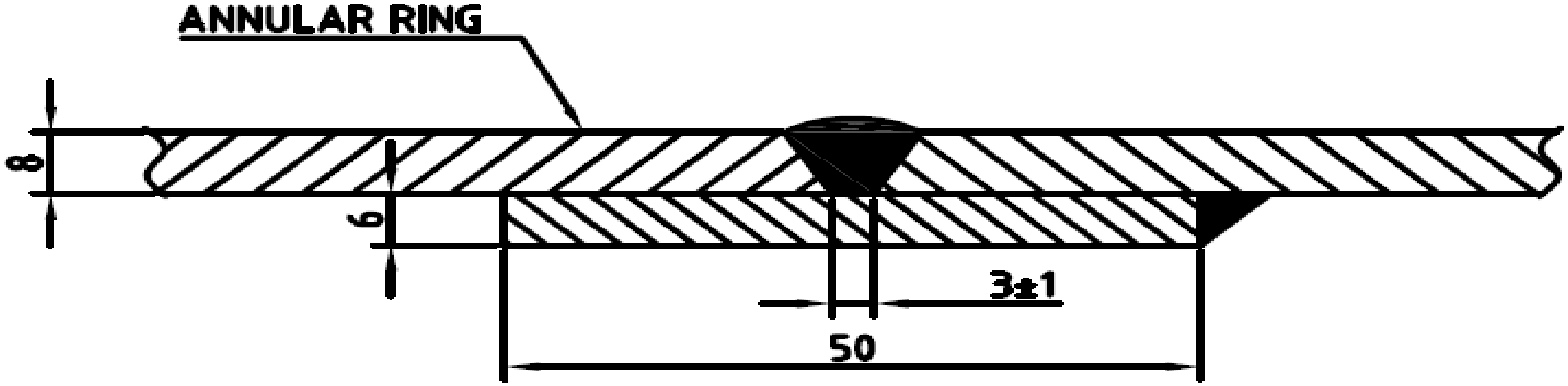


BOTTOM PLATE-9 (BT-9)

SC. = 1 ; 80


0°
8.6°
9.4°
9°
D3
319°
SUMP HOLE
SEE NOTE 3
Ø32136
R13900
Ø1280
TYP.
"A"
Ann-1
Ann-2
Ann-3
BT-1
BT-3
BT-4
BT-5
BT-6
BT-7
BT-8
BT-9
BT-10
BT-11
BT-12
BT-13
BT-14
B
R13900
Ø1280


ANNULAR RING
8
6
3±1
50

MATERIAL AND IMPACT TEST

MATERIALS

- Materials used in the construction of tanks shall conform to the specifications listed in section 4, subject to the modifications and limitations indicated in API 650 standard. Material produced to specifications other than those listed in this section may be employed, provided that the material is certified to meet all of the requirements of an applicable material specification listed in API650 standard and the material's use is approved by the Purchaser. The Manufacturer's proposal shall identify the material specifications to be used. Some listed materials in section 4 are as stated following:

MATERIALS

ASTM PLATES (MOST USEFUL ITEMS):

- ASTM A36M/A36 for plates to a maximum thickness of 40 mm
- ASTM A283M/A283, Grade C, for plates to a maximum thickness of 25 mm
- ASTM A285M/A285, Grade C, for plates to a maximum thickness of 25 mm
- ASTM A516M Grades 380, 415, 450, 485/A516, Grades 55, 60, 65, and 70, for plates to a maximum thickness of 40 mm (insert plates and flanges to a maximum thickness of 100 mm
- ASTM A537M/A537, Class 1 and Class 2, for plates to a maximum thickness of 45 mm (insert plates to a maximum thickness of 100 mm)
- ASTM A573M Grades 400, 450, 485/A573, Grades 58, 65, and 70, for plates to a maximum thickness of 40 mm.

MATERIALS

ASTM SHEETS:

- ASTM A1011M, Grade 33

STRUCTURAL SHAPES

- ASTM A36M/A36
- ASTM A131M/A131
- Structural Steels listed in AISC, Manual of Steel Construction
- EN 10025, Grade S275, Qualities JR, J0, and J2

MATERIALS

- ✓ Piping

API Spec 5L, Grades A, B, and X42

ASTM A53M/A53, Grades A and B

ASTM A106 M/A106, Grades A and B

ASTM A333M/A333, Grades 1 and 6

ASTM A334M/A334, Grades 1 and 6

ASTM A420M/A420, Grade WPL6

- ✓ Forgings :

ASTM A105M/A105
ASTM A181M/A181
ASTM A350M/A350, Grades LF1 and LF2

IMPACT TEST

- Toughness: Toughness is, broadly, a measure of the amount of energy required to cause an item - a test piece or a bridge or a pressure vessel - to fracture and fail. The more energy that is required then the tougher the material. So, The ability of a material to withstand an impact blow is referred to as notch toughness.

- context of an impact test: a measure of the metal's resistance to brittle or fast fracture in the presence of a flaw or notch and fast loading conditions

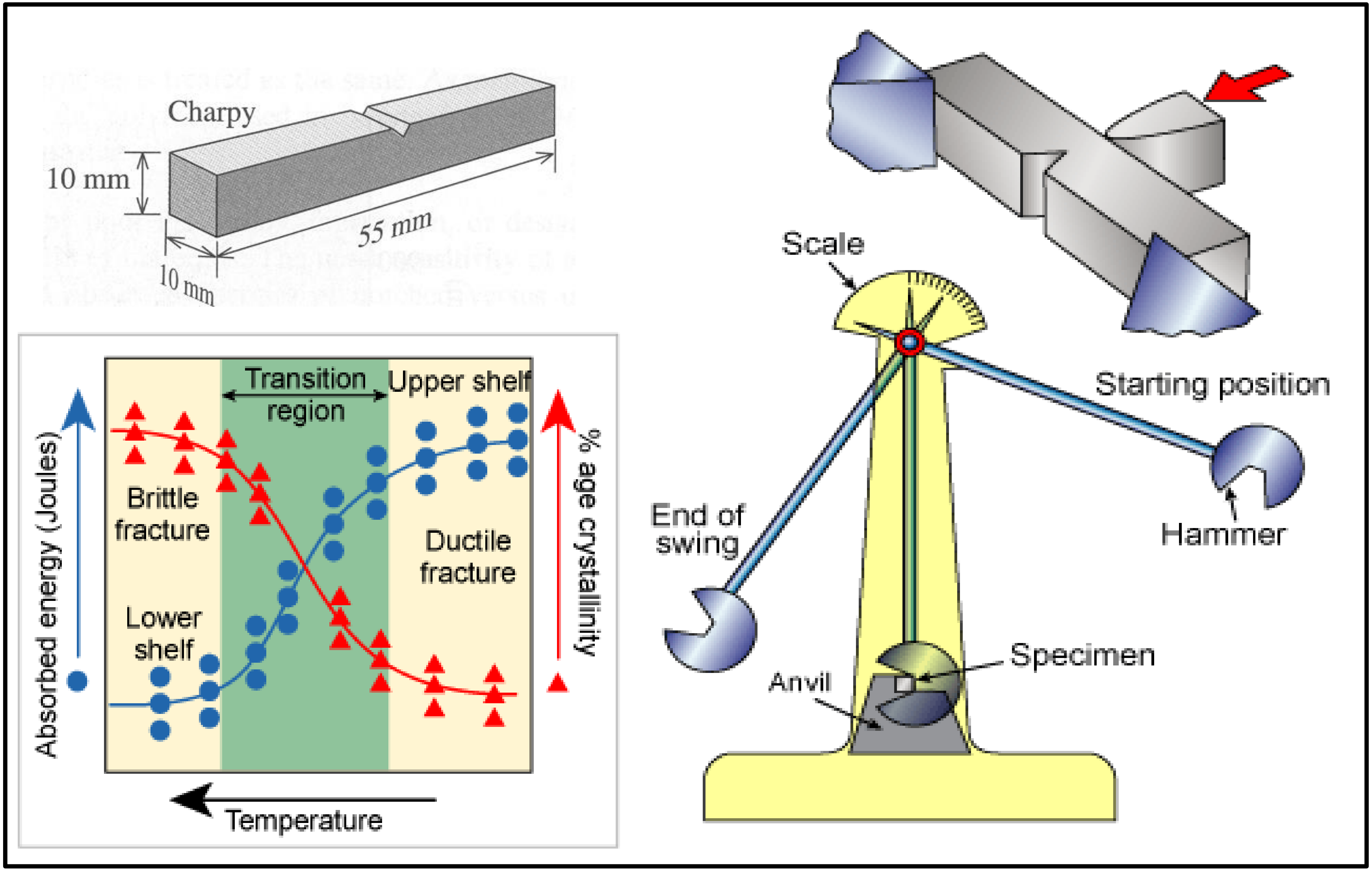
- There are two main forms of impact test, the Izod and the Charpy test. Both involve striking a standard specimen with a controlled weight pendulum travelling at a set speed. The amount of energy absorbed in fracturing the test piece is measured and this gives an indication of the notch toughness of the test material.

IMPACT TEST

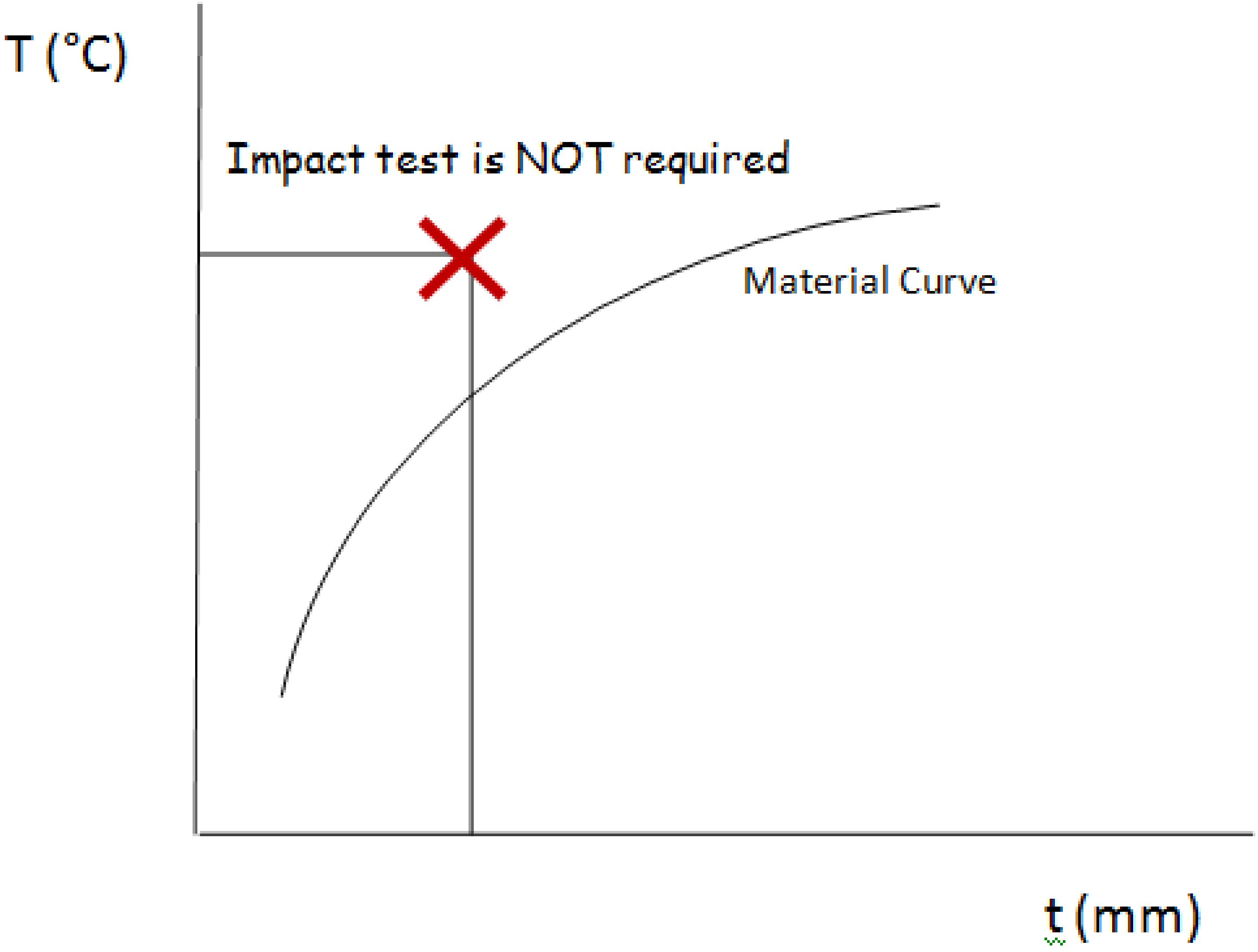
- These tests show that metals can be classified as being either 'brittle' or 'ductile'. A brittle metal will absorb a small amount of energy when impact tested, a tough ductile metal a large amount of energy.

- The energy absorbed is the difference in height between initial and final position of the hammer. The material fractures at the notch and the structure of the cracked surface will help indicate whether it was a brittle or ductile fracture.

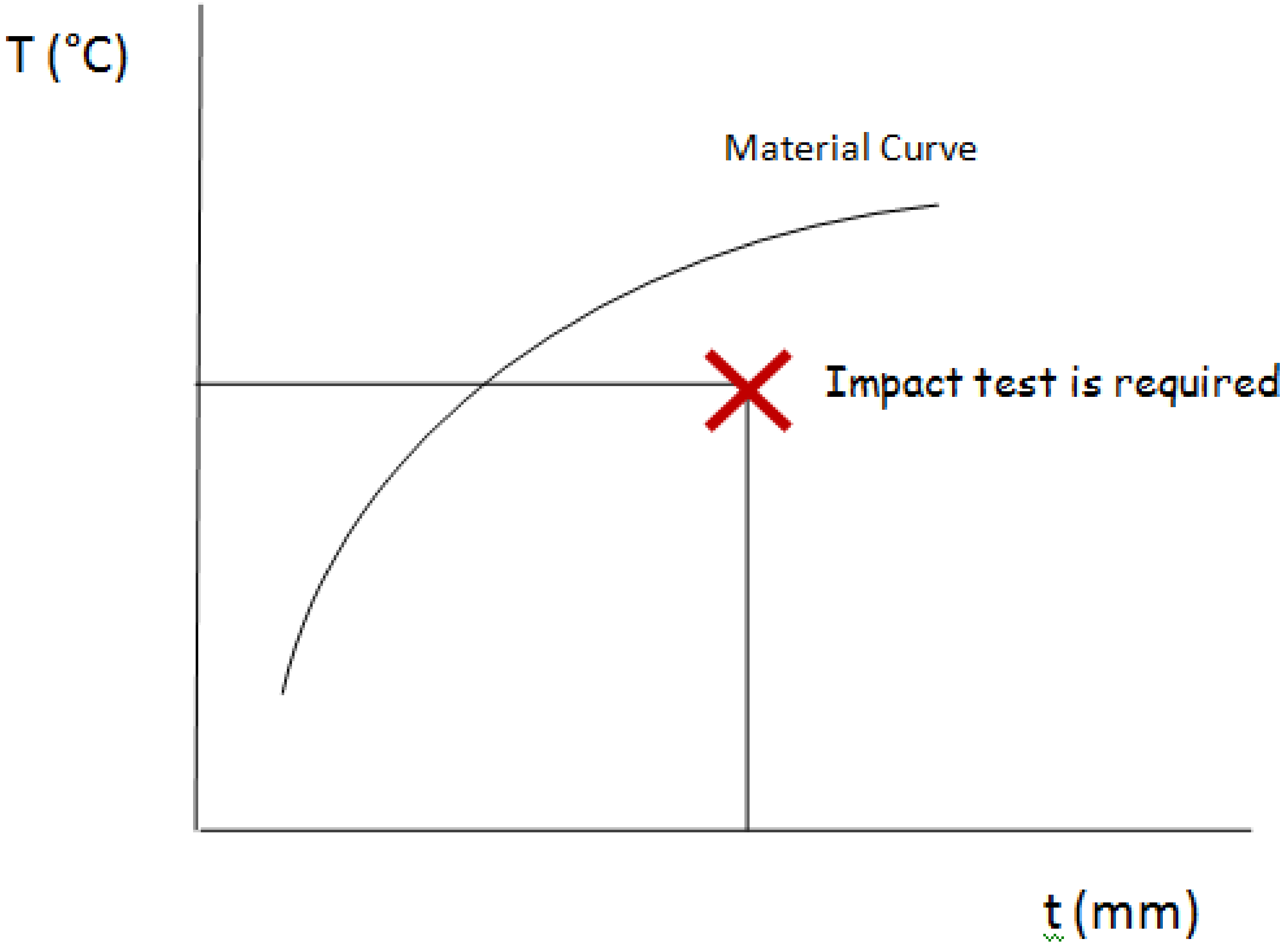
IMPACT TEST


Charpy
10 mm
55 mm
10 mm
Transition region
Upper shelf
Brittle fracture
Lower shelf
Ductile fracture
Absorbed energy (Joules)
% age crystallinity
Temperature
Scale
Starting position
End of swing
Hammer
Specimen
Anvil

IMPACT TEST

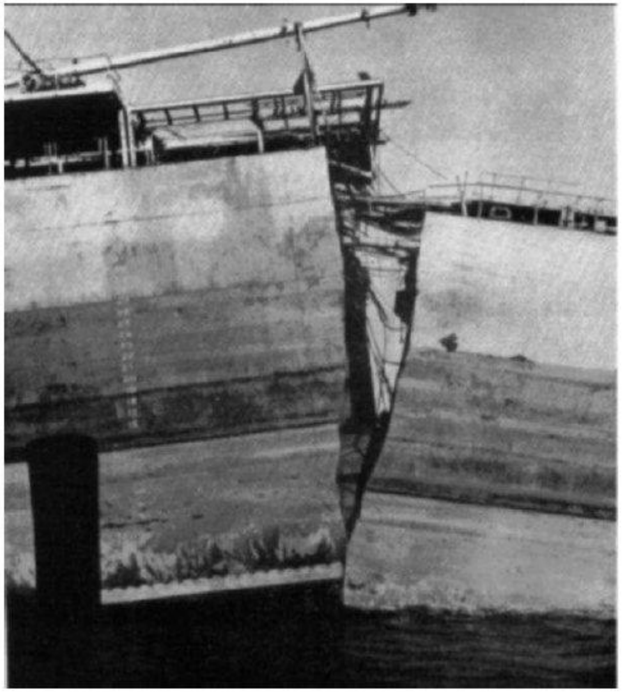


IMPACT TEST



Brittle Fracture

- Failure of Liberty ships in WW II - Low-carbon steels were ductile at RT tensile tests, they became brittle when exposed to lower-temperature ocean environmets.The ships were built and used in the Pacific Ocean but when they were employed in the Atlantic Ocean, which is colder, the ship's material underwent a ductile to brittle transition.



IMPACT TEST

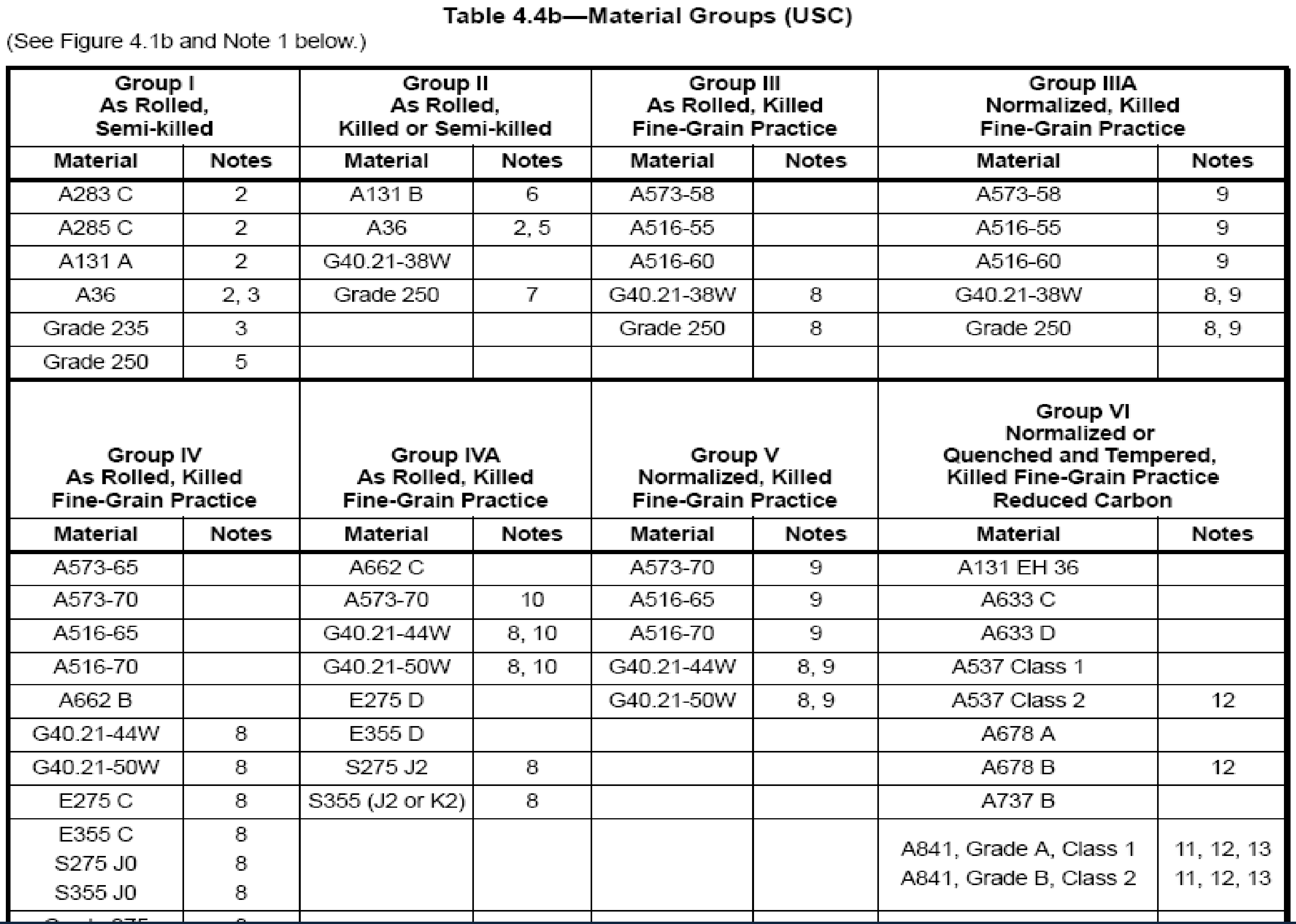


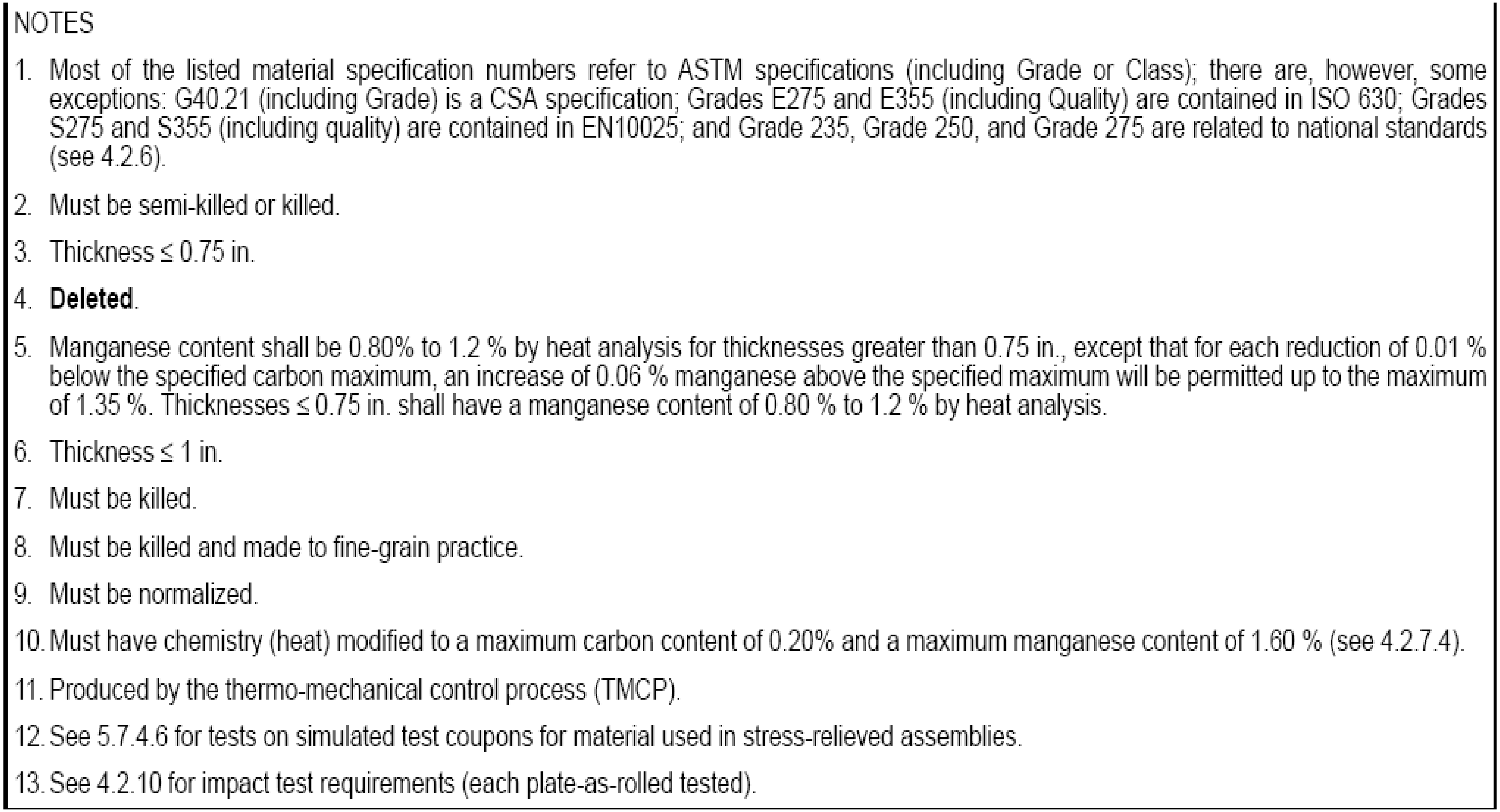
Table 4.4b—Material Groups (USC)

(See Figure 4.1b and Note 1 below.)

Group I As Rolled, Semi-killed		Group II As Rolled, Killed or Semi-killed		Group III As Rolled, Killed Fine-Grain Practice		Group IIIA Normalized, Killed Fine-Grain Practice	
Material	**Notes**	**Material**	**Notes**	**Material**	**Notes**	**Material**	**Notes**
A283 C	2	A131 B	6	A573-58		A573-58	9
A285 C	2	A36	2, 5	A516-55		A516-55	9
A131 A	2	G40.21-38W		A516-60		A516-60	9
A36	2, 3	Grade 250	7	G40.21-38W	8	G40.21-38W	8, 9
Grade 235	3			Grade 250	8	Grade 250	8, 9
Grade 250	5						

Group IV As Rolled, Killed Fine-Grain Practice		Group IVA As Rolled, Killed Fine-Grain Practice		Group V Normalized, Killed Fine-Grain Practice		Group VI Normalized or Quenched and Tempered, Killed Fine-Grain Practice Reduced Carbon	
Material	**Notes**	**Material**	**Notes**	**Material**	**Notes**	**Material**	**Notes**
A573-65		A662 C		A573-70	9	A131 EH 36	
A573-70		A573-70	10	A516-65	9	A633 C	
A516-65		G40.21-44W	8, 10	A516-70	9	A633 D	
A516-70		G40.21-50W	8, 10	G40.21-44W	8, 9	A537 Class 1	
A662 B		E275 D		G40.21-50W	8, 9	A537 Class 2	12
G40.21-44W	8	E355 D				A678 A	
G40.21-50W	8	S275 J2	8			A678 B	12
E275 C	8	S355 (J2 or K2)	8			A737 B	
E355 C S275 J0 S355 J0	8 8 8					A841, Grade A, Class 1 A841, Grade B, Class 2	11, 12, 13 11, 12, 13

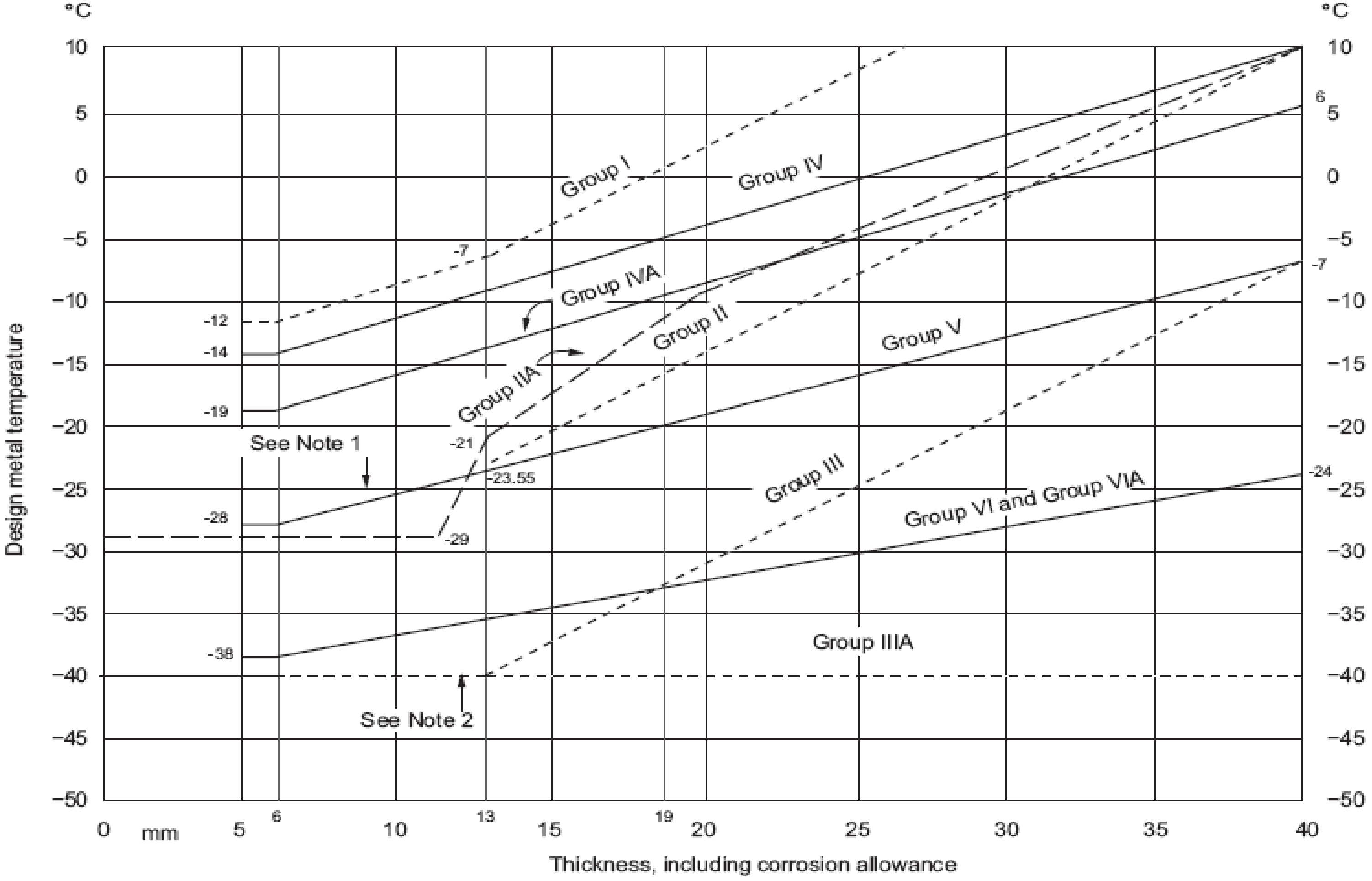
IMPACT TEST



NOTES

1. Most of the listed material specification numbers refer to ASTM specifications (including Grade or Class); there are, however, some exceptions: G40.21 (including Grade) is a CSA specification; Grades E275 and E355 (including Quality) are contained in ISO 630; Grades S275 and S355 (including quality) are contained in EN10025; and Grade 235, Grade 250, and Grade 275 are related to national standards (see 4.2.6).
2. Must be semi-killed or killed.
3. Thickness ≤ 0.75 in.
4. **Deleted**.
5. Manganese content shall be 0.80% to 1.2 % by heat analysis for thicknesses greater than 0.75 in., except that for each reduction of 0.01 % below the specified carbon maximum, an increase of 0.06 % manganese above the specified maximum will be permitted up to the maximum of 1.35 %. Thicknesses ≤ 0.75 in. shall have a manganese content of 0.80 % to 1.2 % by heat analysis.
6. Thickness ≤ 1 in.
7. Must be killed.
8. Must be killed and made to fine-grain practice.
9. Must be normalized.
10. Must have chemistry (heat) modified to a maximum carbon content of 0.20% and a maximum manganese content of 1.60 % (see 4.2.7.4).
11. Produced by the thermo-mechanical control process (TMCP).
12. See 5.7.4.6 for tests on simulated test coupons for material used in stress-relieved assemblies.
13. See 4.2.10 for impact test requirements (each plate-as-rolled tested).

IMPACT TEST



NOTE 1 The Group II and Group V lines coincide at thicknesses less than 13 mm.

NOTE 2 The Group III and Group IIIA lines coincide at thicknesses less than 13 mm.

NOTE 3 The materials in each group are listed in Table 4.4a and Table 4.4b.

NOTE 4 **Deleted.**

NOTE 5 Use the Group IIA and Group VIA curves for pipe and flanges (see 4.5.4.2 and 4.5.4.3).

Design Metal Temperature

- DESIGN METAL TEMPERATURE:
- The lowest temperature considered in the design, which, unless experience or special local conditions justify another assumption, shall be assumed to be 8 °C (15 °F) above the lowest one-day mean ambient temperature of the locality where the tank is to be installed. Isothermal lines of lowest one-day mean temperature are shown in Figure 4.2. The temperatures are not related to refrigerated-tank temperatures (see 1.1.1).

Example

- Min. Amb. Temperature : -15 °C

Course #	Material	Thickness (mm)	Impact Test
1	A 516 70 N	26	?
2	A 516 70	26	?
3	A 516 70	20	?
4	A 283 C	20	?
5	A 283 C	14	?
6	A 283 C	10	?
7	A 283 C	6	?

Example

- Min. Amb. Temperature : -15 °C

Course #	Material	Thickness (mm)	Impact Test
1	A 516 70 N	26	No
2	A 516 70	26	Yes
3	A 516 70	20	Yes
4	A 516 60	20	No
5	A 283 C	20	Yes
6	A 283 C	10	No
7	A 283 C	6	No

IMPACT TEST

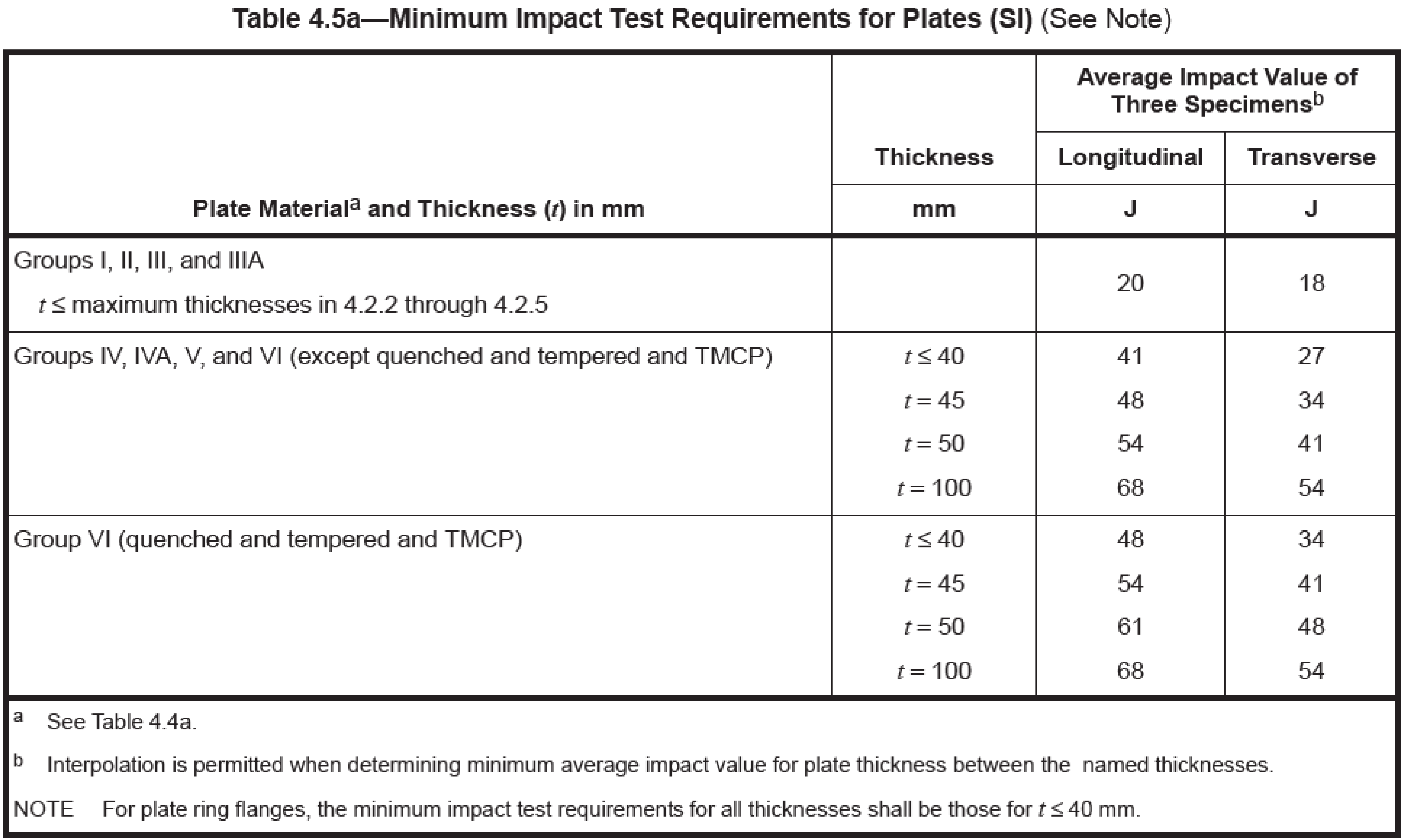


Table 4.5a—Minimum Impact Test Requirements for Plates (SI) (See Note)

Plate Material[a] and Thickness (t) in mm	Thickness	Average Impact Value of Three Specimens[b]	
		Longitudinal	Transverse
	mm	J	J
Groups I, II, III, and IIIA $t \leq$ maximum thicknesses in 4.2.2 through 4.2.5		20	18
Groups IV, IVA, V, and VI (except quenched and tempered and TMCP)	$t \leq 40$	41	27
	$t = 45$	48	34
	$t = 50$	54	41
	$t = 100$	68	54
Group VI (quenched and tempered and TMCP)	$t \leq 40$	48	34
	$t = 45$	54	41
	$t = 50$	61	48
	$t = 100$	68	54

[a] See Table 4.4a.

[b] Interpolation is permitted when determining minimum average impact value for plate thickness between the named thicknesses.

NOTE For plate ring flanges, the minimum impact test requirements for all thicknesses shall be those for $t \leq 40$ mm.

Research case

- As Rolled
- Semi-Killed
- Killed
- Fine-Grain Practice
- Normalized
- Quenched and Tempered

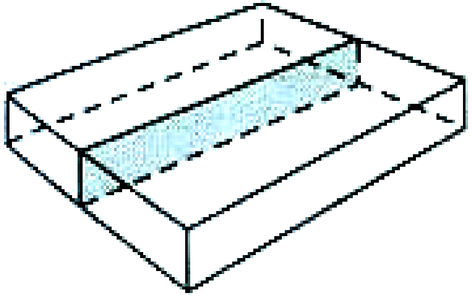
WELDING

WELDING

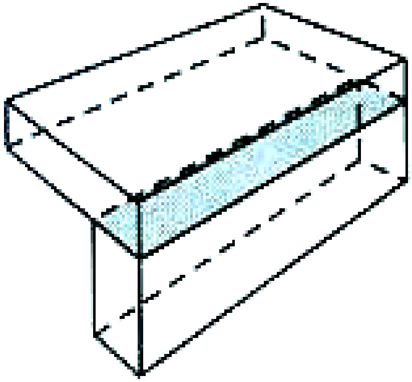
TYPE OF JOINTS:

1. Butt joint (لب به لب)
2. Corner joint (گوشه ای)
3. T-joint (سپری)
4. Lap joint (لبه روی هم)
5. Edge joint (لبه ای)

WELDING

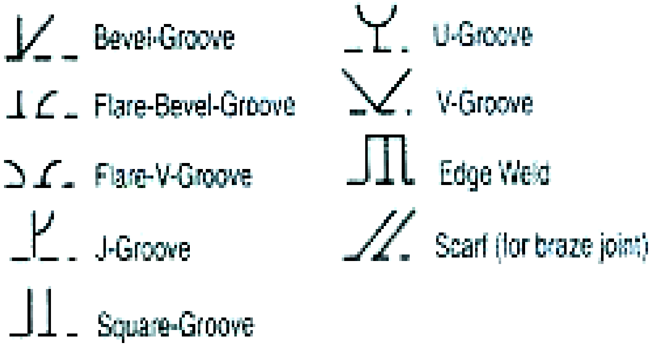


(A) BUTT JOINT



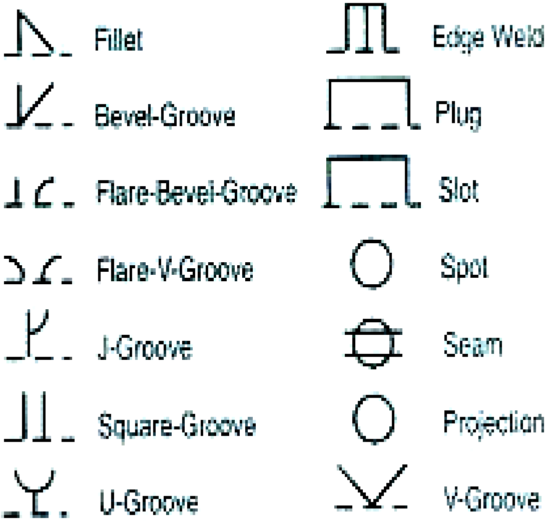
(B) CORNER JOINT

APPLICABLE WELDS and WELD SYMBOL



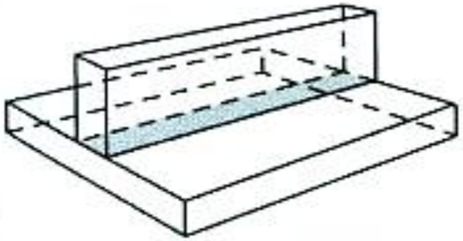
- Bevel-Groove
- Flare-Bevel-Groove
- Flare-V-Groove
- J-Groove
- Square-Groove
- U-Groove
- V-Groove
- Edge Weld
- Scarf (for braze joint)

APPLICABLE WELDS and WELD SYMBOL



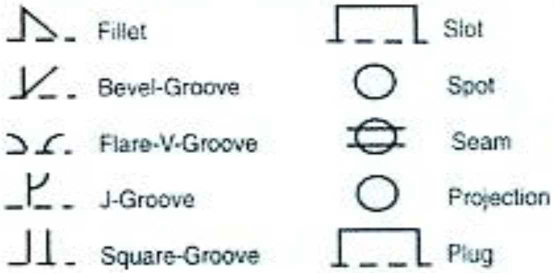
- Fillet
- Bevel-Groove
- Flare-Bevel-Groove
- Flare-V-Groove
- J-Groove
- Square-Groove
- U-Groove
- Edge Weld
- Plug
- Slot
- Spot
- Seam
- Projection
- V-Groove

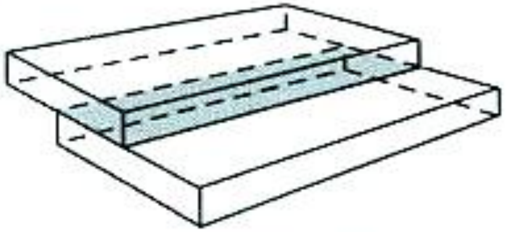
WELDING



(C) T-JOINT

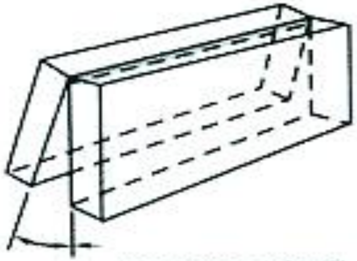
APPLICABLE WELDS and WELD SYMBOL





(D) LAP JOINT

APPLICABLE WELDS and WELD SYMBOL



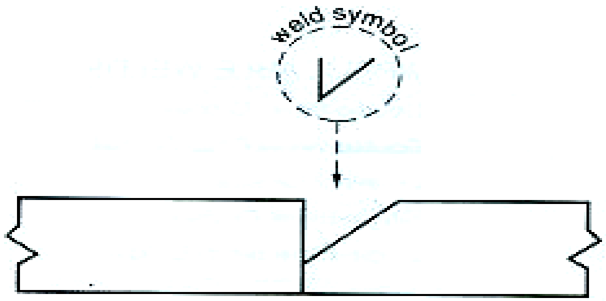
(E) EDGE JOINT

APPLICABLE WELDS and WELD SYMBOL

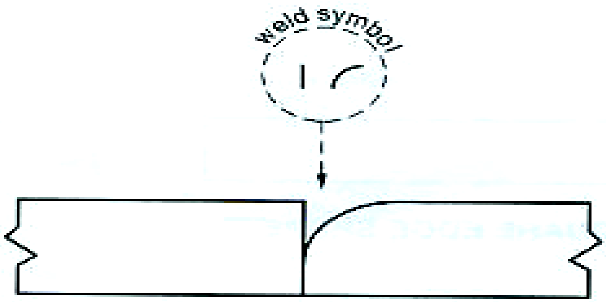
WELDING

TYPE OF GROOVE:

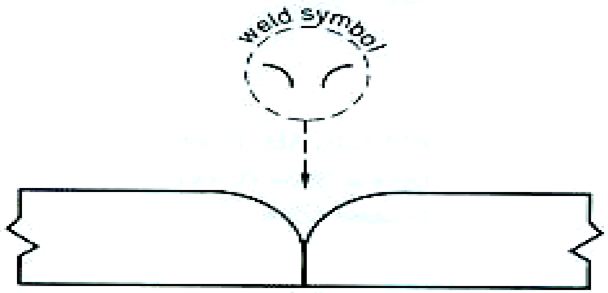
Butt joint



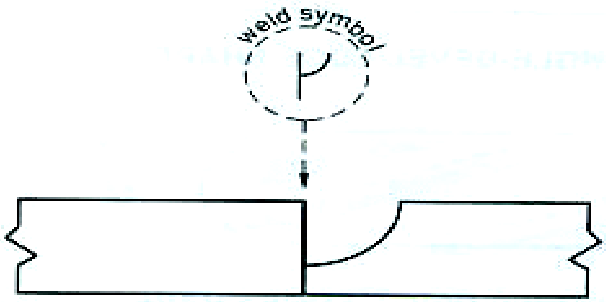
Bevel-Groove



Flare-Bevel-Groove



Flare-V-Groove

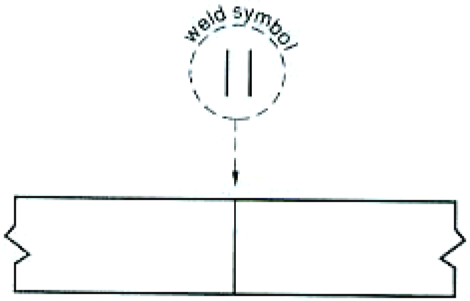


J-Groove

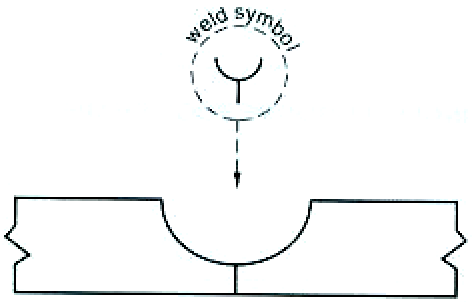
WELDING

TYPE OF GROOVE:

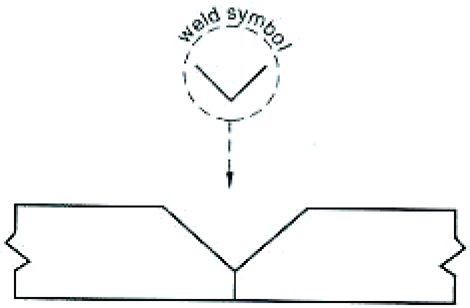
Butt joint



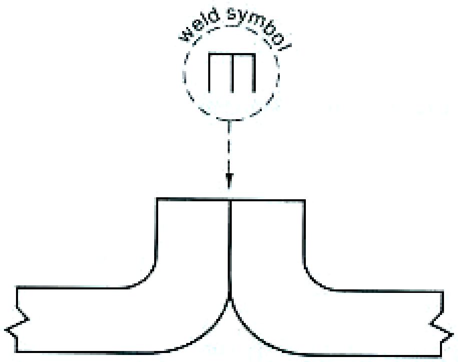
Square-Groove



U-Groove



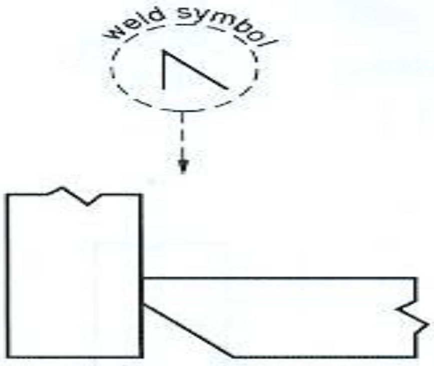
V-Groove



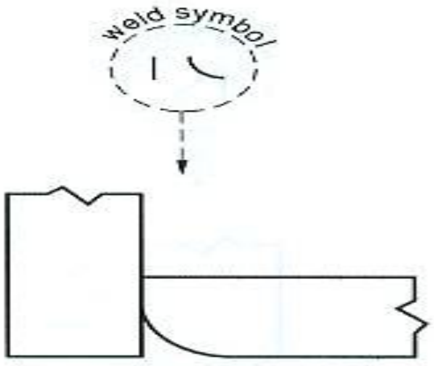
Edge

WELDING

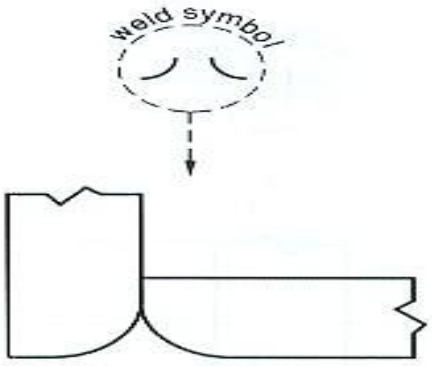
Corner joint (گوشه ای)



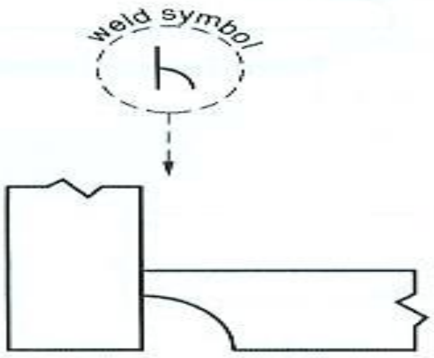
Bevel-Groove



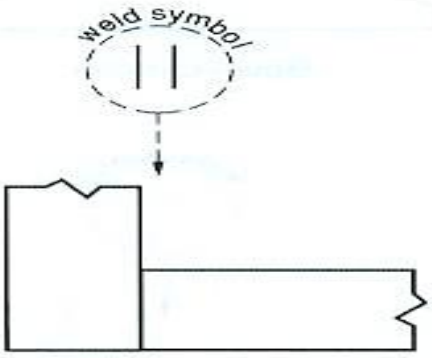
Flare-Bevel-Groove



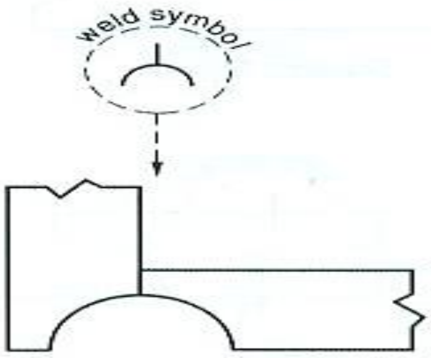
Flare-V-Groove



J-Groove

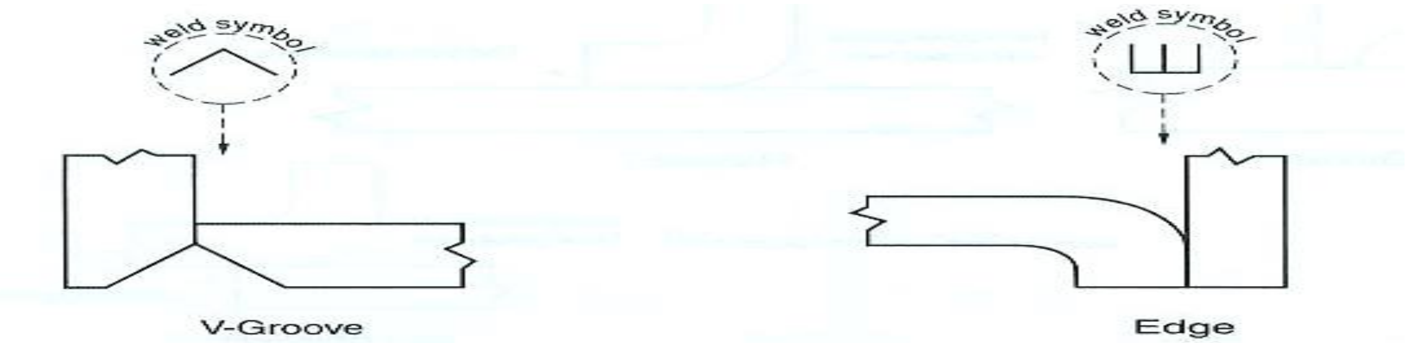


Square-Groove

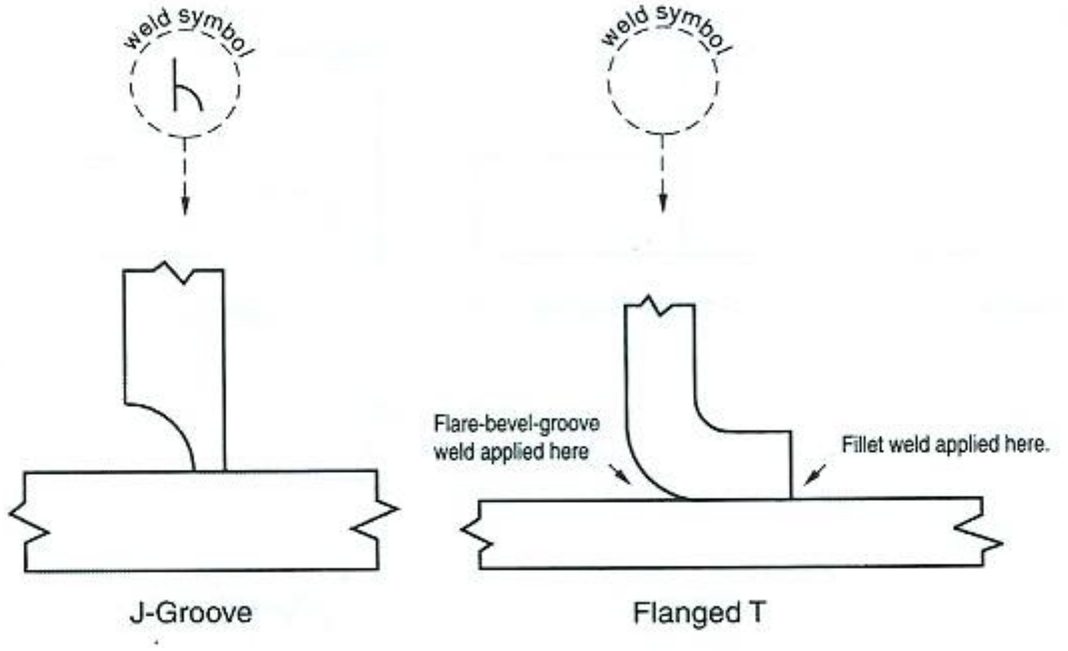


U-Groove

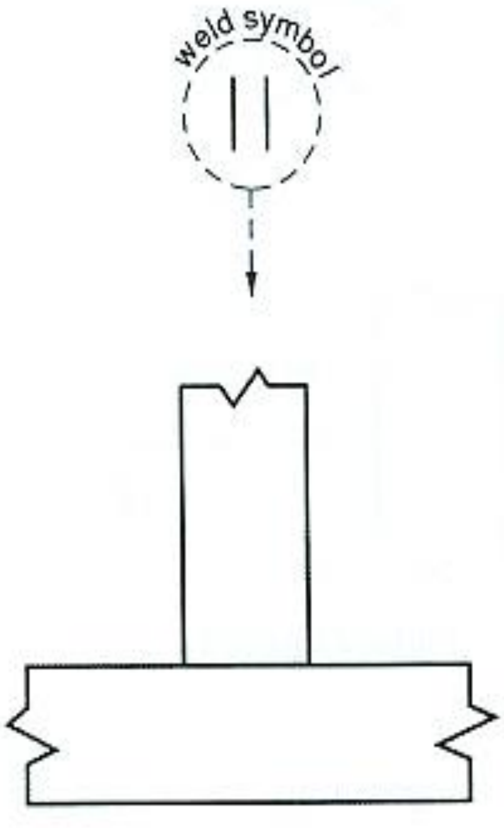
WELDING



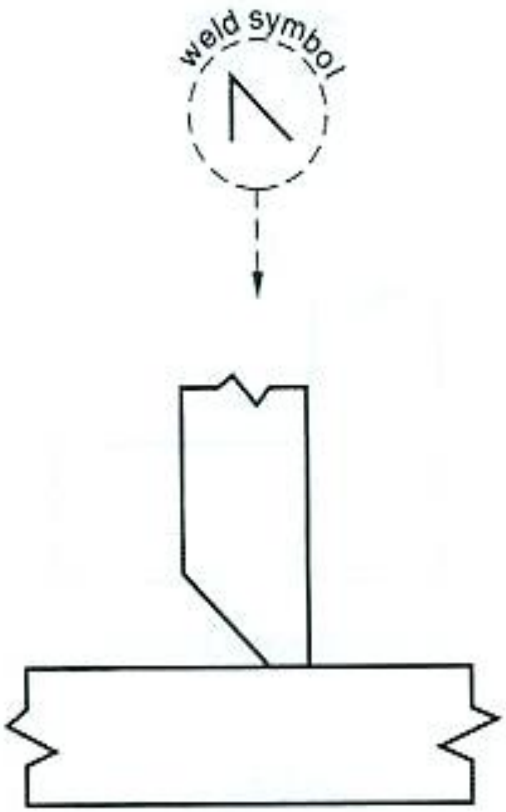
T-joint (سپری)



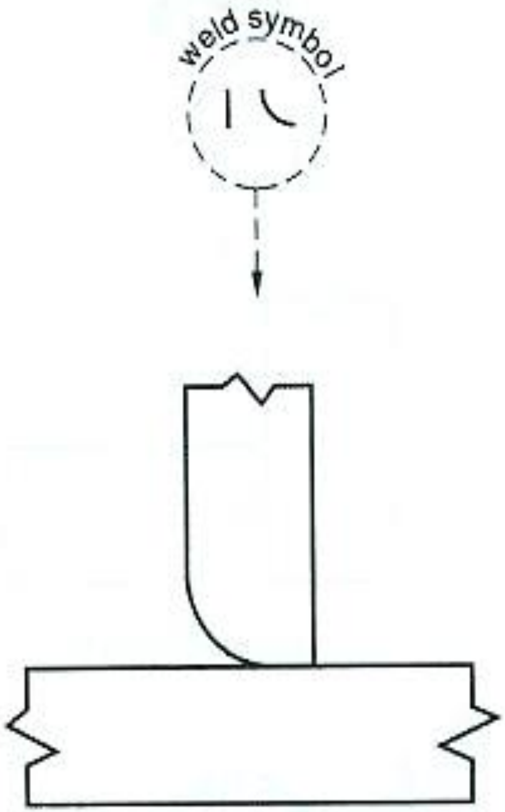
WELDING



Square-Groove



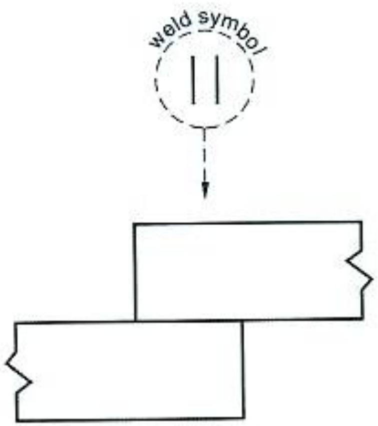
Bevel-Groove



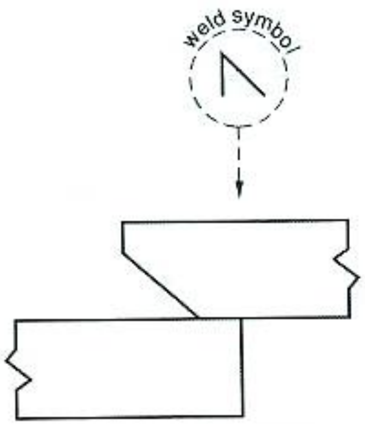
Flare-Bevel-Groove

WELDING

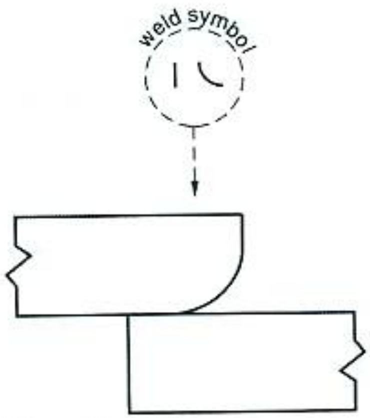
Lap joint (لبه روی هم)



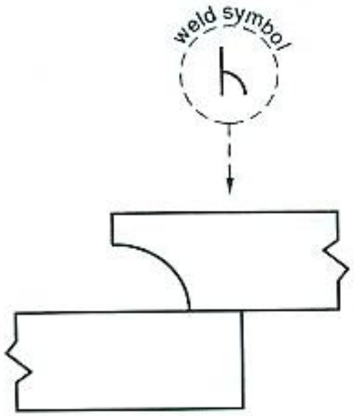
Square-Groove



Bevel-Groove

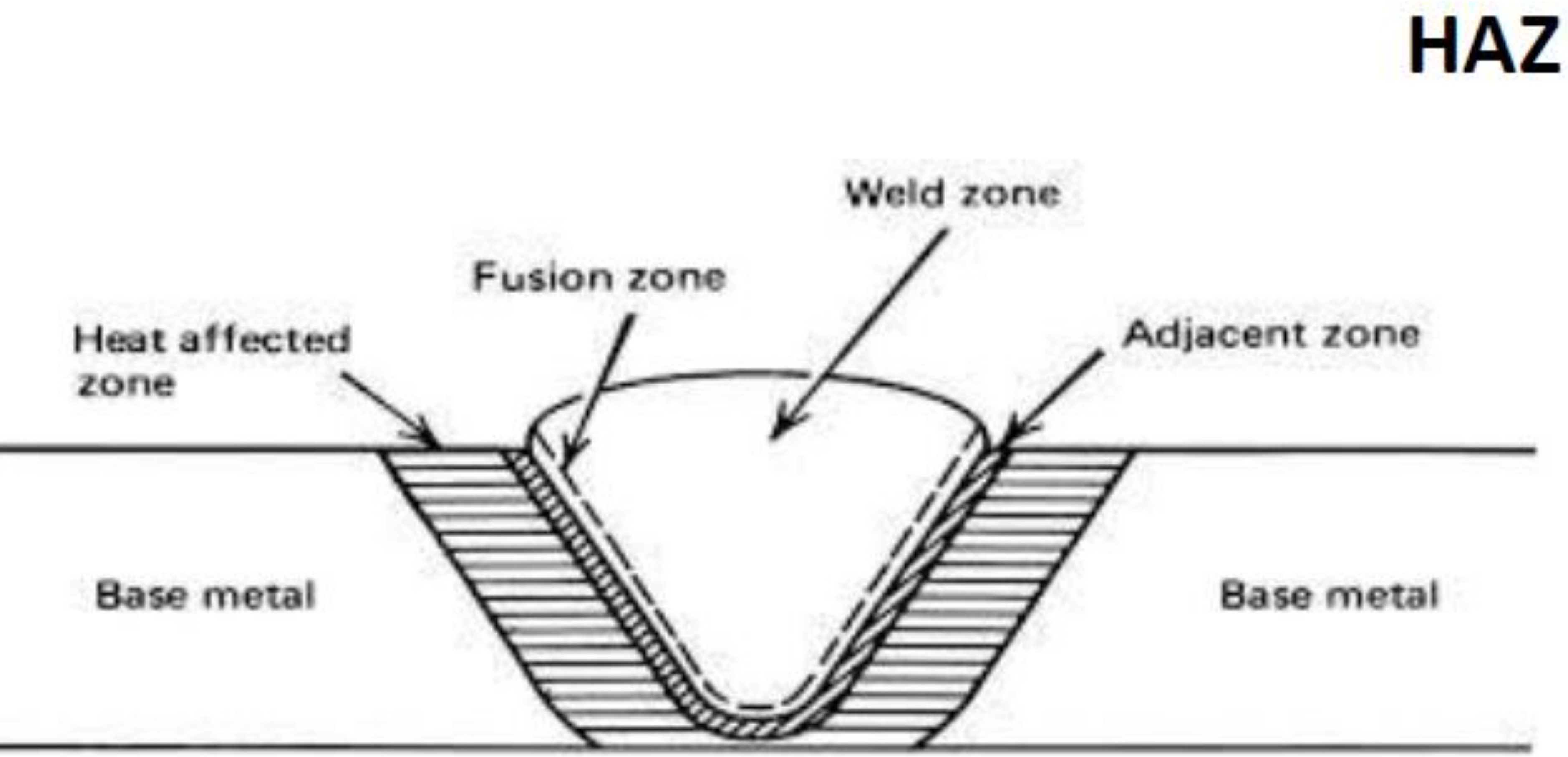


Flare-Bevel-Groove



J-Groove

HAZ AREA


HAZ
Weld zone
Fusion zone
Heat affected zone
Adjacent zone
Base metal
Base metal

HAZ

- ناحیه متأثر از حرارت ((Heat Affected Zone: HAZ) قسمتی از فلز جوش است که در آن اگر چه فلز پایه ذوب نشده است اما ساختار و دانه بندی آن در اثر حرارت ناشی از جوشکاری تغییر یافته است. در پایان فرآیندهای جوشکاری به دلیل سرعت بالای سرد شدن، ساختارهای مارتنزیتی تشکیل میگردد. این نواحی مستعد ایجاد ترک در قطعه جوشکاری شده هستند. وقتی فلزات و آلیاژ هایی که استحاله چند شکلی ندارند مانند مس، نیکل، آلومینیوم، جوش داده میشوند، ریز ساختار در HAZ تغییر نمیکند با این وجود که ممکن است تبلور مجدد یا رشد دانه در آن اتفاق بیفتد. این در حالیست که در فلزات و آلیاژ هایی که استحاله چند شکلی دارند)مانند فولادها(تغییرات ریز ساختاری قابل ملاحظه ای در ناحیه متاثر از حرارت رخ میدهد که این تغییرات خواص مکانیکی و رفتار عملی اتصال جوش را تحت تأثیر قرار میدهد .

HAZ

وسعت و گستردگی ناحیه متاثر از حرارت به عوامل زیر بستگی دارد

- روش جوشکاری
- سرعت جوشکاری
- درجه حرارت پیشگرم
- تعداد پاس های جوشکاری
- ابعاد قطعه
- شکل طرح اتصال
- شکل حوضچه جوش

HAZ

- روش جوشکاری در جوشکاری قوس الکتریکی دستی، وسعت ناحیه HAZ دارای کمترین مقدار است و به 2 تا ۸.2 میلیمتر میرسد. در جوشکاری با الکترودهای پوششدار وسعت این ناحیه ۳ تا ۱۹ میلیمتر است در حالی که در جوشکاری گازی به 2۹ تا 2۸ میلیمتر میرسد. علت این است که در روشهای جوشکاری با قوس الکتریکی، امکان تمرکز حرارت در یک نقطه وجود داشته در حالی که در روشهای گازی، حرارت در سطح توزیع شده و در نواحی اطراف ناحیه متاثر از حرارت گسترش مییابد . جنس فلز پایه : در فلزاتی که ضریب انتقال حرارت)هدایت حرارتی(بالاتری دارند، ایجاد تمرکز حرارت غیرممکن است بنابراین منطقه HAZ در فلزات و آلیاژهای آلومینیوم و مس نسبت به فولادها از وسعت بیشتری برخوردارند. در بین فولادها نیز وسعت ناحیه متاثر از حرارت در فولادهای کربنی بیشتر از سایر فولادها میباشد.
- سرعت جوشکاری : هر چقدر میزان سرعت جوشکاری بالاتر باشد، وسعت ناحیه متاثر از حرارت کوچکتر میگردد؛ زیرا حرارت تولیدی در ناحیه جوش فرصت انتقال به نواحی اطراف و پراکنده شدن را ندارد
- درجه حرارت پیشگرم : هر چه میزان دمای پیشگرم قطعه جوشکاری کمتر باشد، وسعت منطقه HAZ کمتر خواهد شد؛ زیرا چنانچه فلز تا حد قابل ملاحظه ای پیشگرم شود، در واقع هنگام جوشکاری به گرمتر شدن نواحی اطراف جوش کمک شده است
- تعداد پاس های جوشکاری : در جوش تک پاسی، به دلیل اعمال حرارت ورودی بیش از حد و طولانی شدن زمان جوشکاری و همچنین طولانی شدن زمان انجماد، وسعت ناحیه متاثر از حرارت افزایش می یابد

HAZ

- ابعاد قطعه : قطعات ضخیم تر، قدرت جذب حرارت بیشتری داشته و سرعت سرد شدن جوش نیز افزایش می یابد . متغیرهای جوشکاری : متغیرهایی مانند شدت جریان، ولتاژ و قطر الکترود نیز بر وسعت ناحیه HAZ تاثیر میگذارد. زیرا با افزایش شدت جریان، ولتاژ و قطر الکترود وسعت ناحیه HAZ افزایش می یابد .
- شکل طرح اتصال : بطور مثال با مقایسه بین جوش نبشی و جوش لبه ای در صورتی که ضخامت ورق در محل هر دو نوع اتصال با هم برابر باشد، به دلیل سرعت سرد شدن بالاتر در جوش نبشی، وسعت ناحیه متاثر از حرارت در آن کوچکتر از جوش لبه ای میگردد .
- شکل حوضچه جوش: همچنین در دو نوع یکسان جوش نبشی چنانچه گرده جوش در یکی از اتصالات به شکل محدب باشد، سطح تماس جوش با فلز پایه بیشتر شده و در نتیجه حرارت را سریعتر به محیط اطراف منتقل میکند. این امر سبب می شود که وسعت ناحیه HAZ نسبت به گرده مقعر جوش، بیشتر گردد.

منبع: مرکز پژوهش و مهندسی جوش ایران ، ساختار متالورژیکی مقاطع جوشکاری شده
محقق : مهندس ودود عزیزی- کلاس PRESSURE VESSEL سال 93

DESIGN CONSIDERATION

DESIGN (Design Considerations)

✓ 5.2. Design Considerations

a) Dead Load (DL): The weight of the tank or tank component, including any corrosion allowance.

b) Design External Pressure (Pe): Shall not be less than 0.25 kPa, except that Pe shall be considered as 0 kPa

c) Design Internal Pressure (Pi): Shall not exceed 18 kPa.

d) Hydrostatic Test: The load due to filling the tank with water to the design liquid level.

f) Minimum Roof Live Load: 1.0 kPa on the horizontal projected area of the roof.

h) Snow (S)

k) Wind (W)

l) External Loads

DESIGN (Design Considerations)

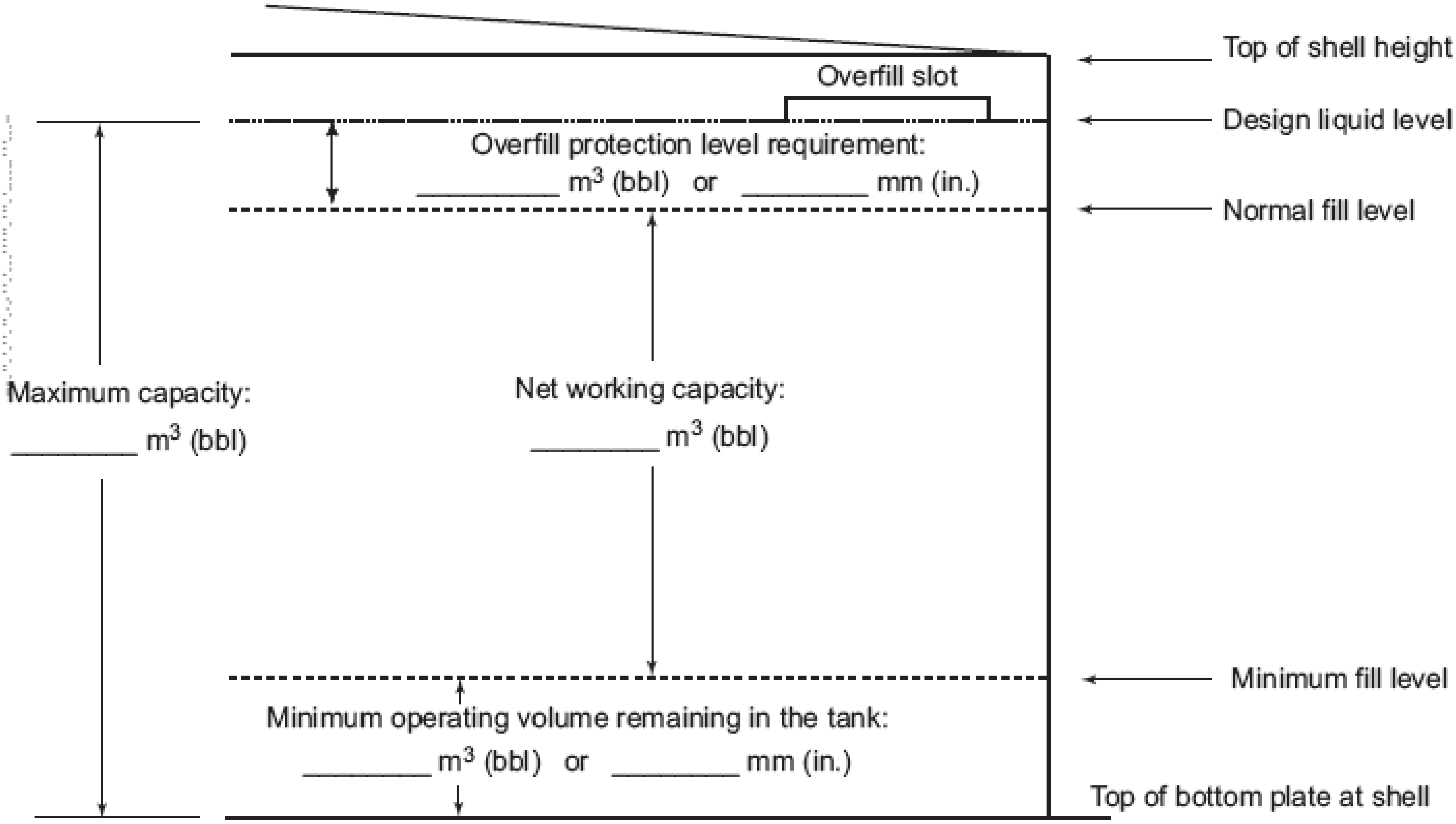


Figure 5.4—Storage Tank

Allowable Stresses

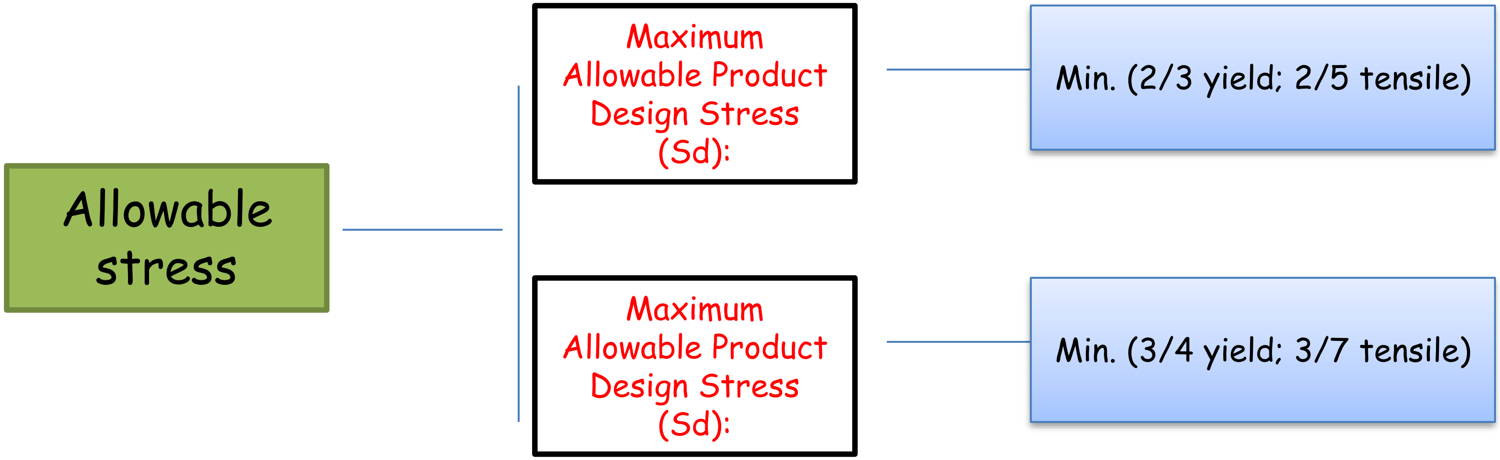
✓ 5.6.2. Allowable Stress

Maximum Allowable Product Design Stress (Sd):

- The design stress basis, Sd, shall be either two-thirds the yield strength or two-fifths the tensile strength, whichever is less

Maximum Allowable hydrostatic test Stress (St):

- The hydrostatic test basis shall be either three-fourths the yield strength or three-sevenths the tensile strength, whichever is less.



Allowable stress

Maximum Allowable Product Design Stress (Sd):

Min. (2/3 yield; 2/5 tensile)

Maximum Allowable Product Design Stress (Sd):

Min. (3/4 yield; 3/7 tensile)

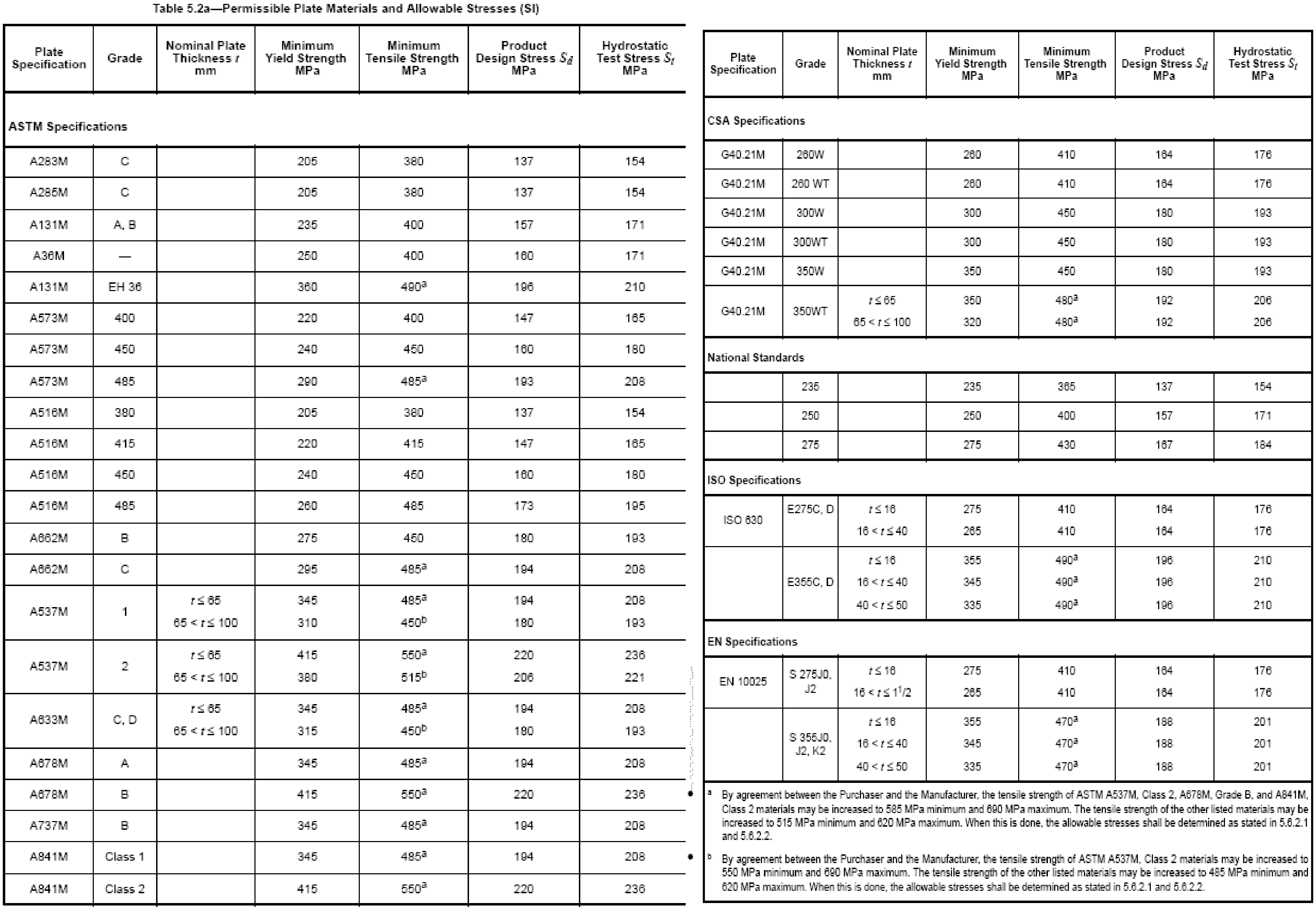


Table 5.2a—Permissible Plate Materials and Allowable Stresses (SI)

Plate Specification	Grade	Nominal Plate Thickness t mm	Minimum Yield Strength MPa	Minimum Tensile Strength MPa	Product Design Stress S_d MPa	Hydrostatic Test Stress S_t MPa
ASTM Specifications						
A283M	C		205	380	137	154
A285M	C		205	380	137	154
A131M	A, B		235	400	157	171
A36M	—		250	400	160	171
A131M	EH 36		360	490[a]	196	210
A573M	400		220	400	147	165
A573M	450		240	450	160	180
A573M	485		290	485[a]	193	208
A516M	380		205	380	137	154
A516M	415		220	415	147	165
A516M	450		240	450	160	180
A516M	485		260	485	173	195
A662M	B		275	450	180	193
A662M	C		295	485[a]	194	208
A537M	1	$t \le 65$ $65 < t \le 100$	345 310	485[a] 450[b]	194 180	208 193
A537M	2	$t \le 65$ $65 < t \le 100$	415 380	550[a] 515[b]	220 206	236 221
A633M	C, D	$t \le 65$ $65 < t \le 100$	345 315	485[a] 450[b]	194 180	208 193
A678M	A		345	485[a]	194	208
A678M	B		415	550[a]	220	236
A737M	B		345	485[a]	194	208
A841M	Class 1		345	485[a]	194	208
A841M	Class 2		415	550[a]	220	236
CSA Specifications						
G40.21M	260W		260	410	164	176
G40.21M	260 WT		260	410	164	176
G40.21M	300W		300	450	180	193
G40.21M	300WT		300	450	180	193
G40.21M	350W		350	450	180	193
G40.21M	350WT	$t \le 65$ $65 < t \le 100$	350 320	480[a] 480[a]	192 192	206 206
National Standards						
	235		235	365	137	154
	250		250	400	157	171
	275		275	430	167	184
ISO Specifications						
ISO 630	E275C, D	$t \le 16$ $16 < t \le 40$	275 265	410 410	164 164	176 176
	E355C, D	$t \le 16$ $16 < t \le 40$ $40 < t \le 50$	355 345 335	490[a] 490[a] 490[a]	196 196 196	210 210 210
EN Specifications						
EN 10025	S 275J0, J2	$t \le 16$ $16 < t \le 1^{1}/2$	275 265	410 410	164 164	176 176
	S 355J0, J2, K2	$t \le 16$ $16 < t \le 40$ $40 < t \le 50$	355 345 335	470[a] 470[a] 470[a]	188 188 188	201 201 201

[a] By agreement between the Purchaser and the Manufacturer, the tensile strength of ASTM A537M, Class 2, A678M, Grade B, and A841M, Class 2 materials may be increased to 585 MPa minimum and 690 MPa maximum. The tensile strength of the other listed materials may be increased to 515 MPa minimum and 620 MPa maximum. When this is done, the allowable stresses shall be determined as stated in 5.6.2.1 and 5.6.2.2.

[b] By agreement between the Purchaser and the Manufacturer, the tensile strength of ASTM A537M, Class 2 materials may be increased to 550 MPa minimum and 690 MPa maximum. The tensile strength of the other listed materials may be increased to 485 MPa minimum and 620 MPa maximum. When this is done, the allowable stresses shall be determined as stated in 5.6.2.1 and 5.6.2.2.

Required Calculation

- Thicknesses
- Attachments
- Wind and stability
- Seismic
- Internal pressure
- External pressure

Thickness Calculation

- Bottom and annular plate thickness calculation
- Shell plate thickness calculation
- Roof plate thickness calculation

SHELL DESIGN

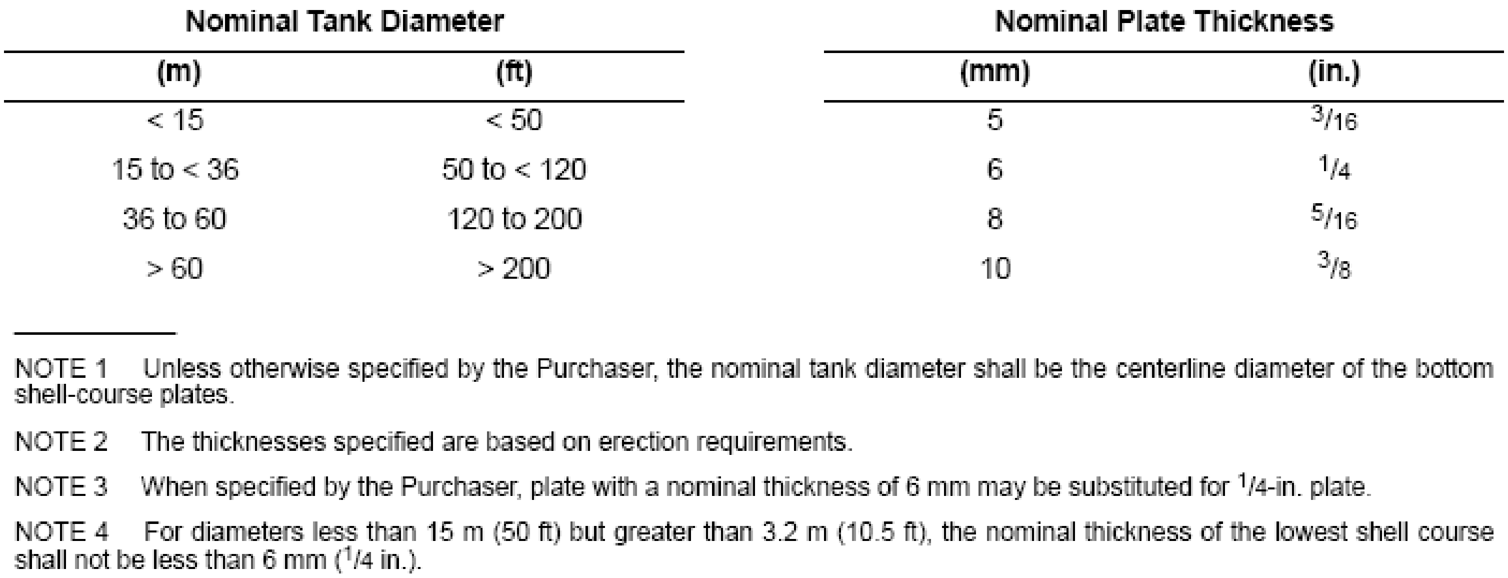
DESIGN (SHELL DESIGN)

- Liquid levels

DESIGN (SHELL DESIGN)

5.6. SHELL DESIGN

- 5.6.1.1 The required shell thickness shall be the greater of the design shell thickness, including any corrosion allowance, or the hydrostatic test shell thickness, but the shell thickness shall not be less than the following:



Nominal Tank Diameter		Nominal Plate Thickness	
(m)	(ft)	(mm)	(in.)
< 15	< 50	5	${}^{3}/_{16}$
15 to < 36	50 to < 120	6	${}^{1}/_{4}$
36 to 60	120 to 200	8	${}^{5}/_{16}$
> 60	> 200	10	${}^{3}/_{8}$

- NOTE 1 Unless otherwise specified by the Purchaser, the nominal tank diameter shall be the centerline diameter of the bottom shell-course plates.

NOTE 2 The thicknesses specified are based on erection requirements.

- NOTE 3 When specified by the Purchaser, plate with a nominal thickness of 6 mm may be substituted for ${}^{1}/_{4}$-in. plate.

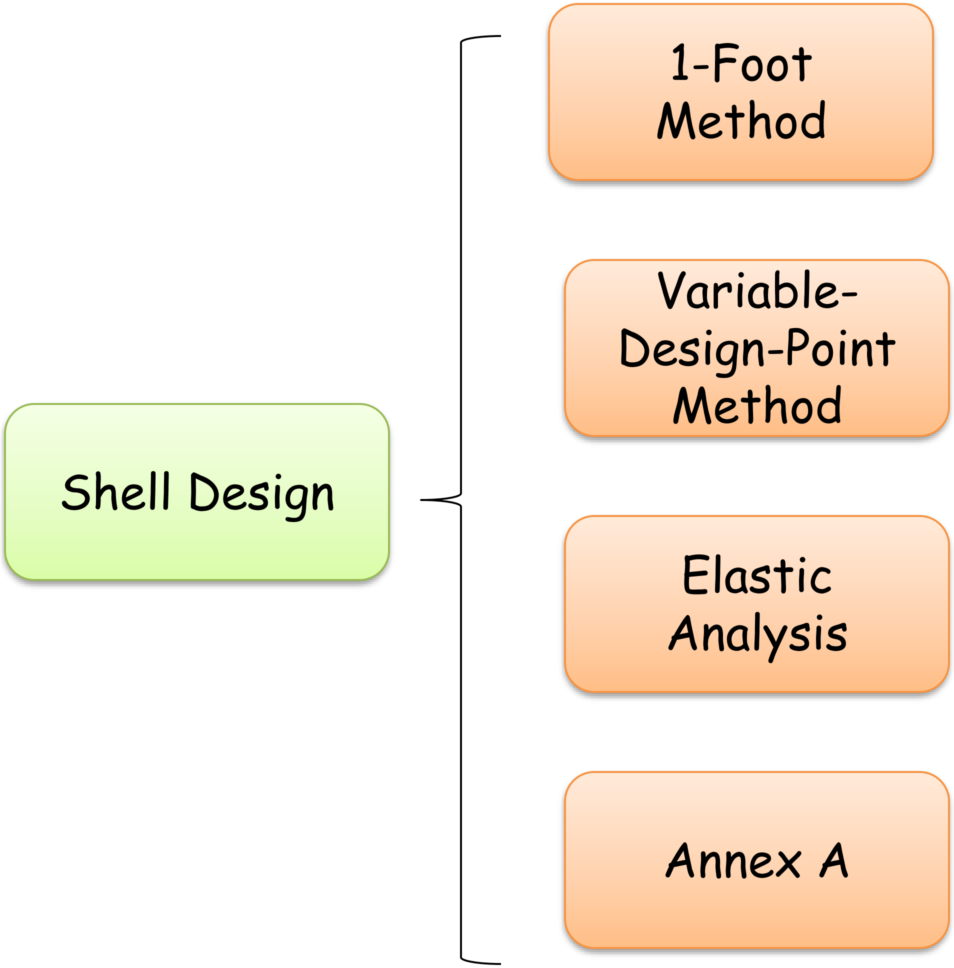
NOTE 4 For diameters less than 15 m (50 ft) but greater than 3.2 m (10.5 ft), the nominal thickness of the lowest shell course shall not be less than 6 mm (${}^{1}/_{4}$ in.).

DESIGN (SHELL DESIGN)

- 5.6.1.2 Unless otherwise agreed to by the Purchaser, the shell plates shall have a minimum nominal width of 1800 mm (72 in.)

- 5.6.1.3 When the allowable stress for an upper shell course is lower than the allowable stress of the next lower shell course, The lower shell course thickness shall be no less than the thickness required of the upper shell course for product and hydrostatic test loads by 5.6.3 or 5.6.4.

DESIGN (SHELL DESIGN)



Shell Design

1-Foot Method

Variable-Design-Point Method

Elastic Analysis

Annex A

DESIGN (SHELL DESIGN)

- **1-Foot Method :** he 1-foot method calculates the thicknesses required at design points 0.3 m (1 ft) above the bottom of each shell course.

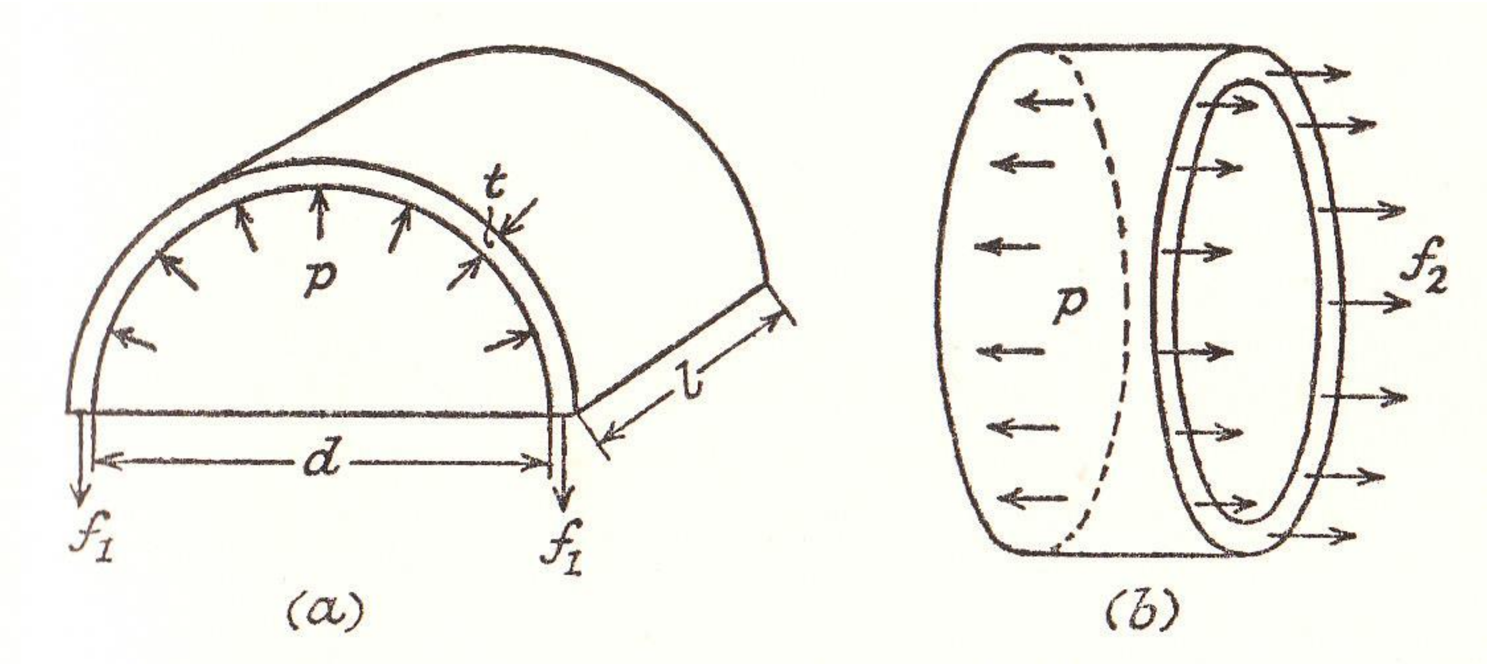
- **Variable-Design-Point Method :**Design by the variable-design-point method gives shell thicknesses at design points that result in the calculated stresses being relatively close to the actual circumferential shell stresses.

- **Elastic Analysis:** For tanks where L/H is greater than 1000/6, the selection of shell thicknesses shall be based on an elastic analysis

- **Annex A :** Annex A permits an alternative shell design with a fixed allowable stress of 145 MPa (21,000 lbf/in.2) and a joint efficiency factor of 0.85 or 0.70. This design may only be used for tanks with shell thicknesses less than or equal to 13 mm

SHELL DESIGN

- Hoop (circumferential) stress :
- This is the stress trying to split the vessel open along its length. Confusingly, this acts on the longitudinal weld seam (if there is one).



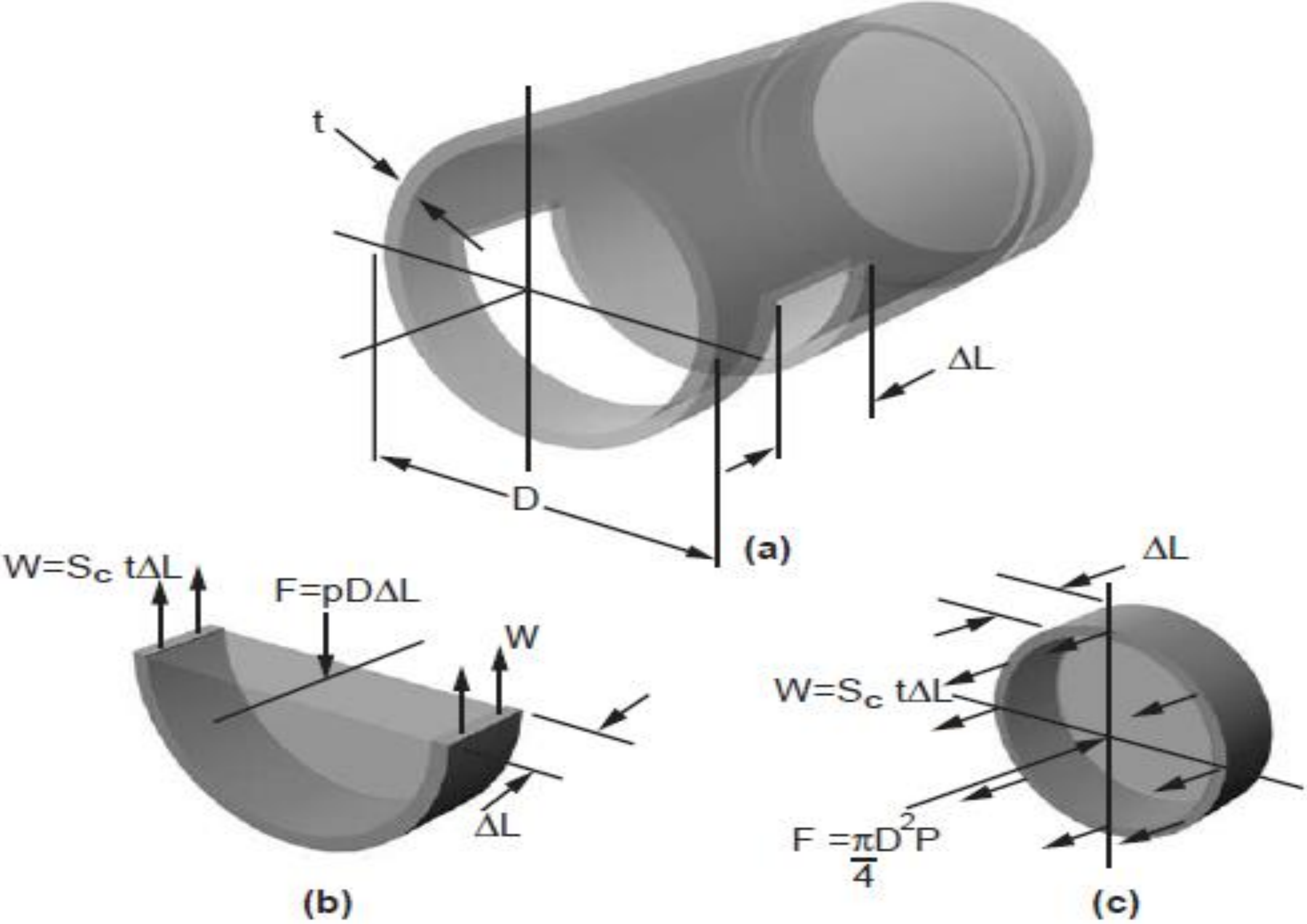


Figure 7.1 Forces and stresses in a pressurized cylinder.

SHELL DESIGN

- circumferential stress :

$$S_C = \frac{RP}{t}$$

where: R = inside radius of the cylinder
t = thickness of cylinder
P = internal pressure

- Longitudinal stress :

$$S_L = \frac{RP}{2t}$$

where: R = inside radius of the cylinder
t = thickness of cylinder
P = internal pressure

SHELL DESIGN

- Liquid static head stress:

I) $S_C = \frac{RP}{t}$

II) $P = \rho g h$

Substituting II in I

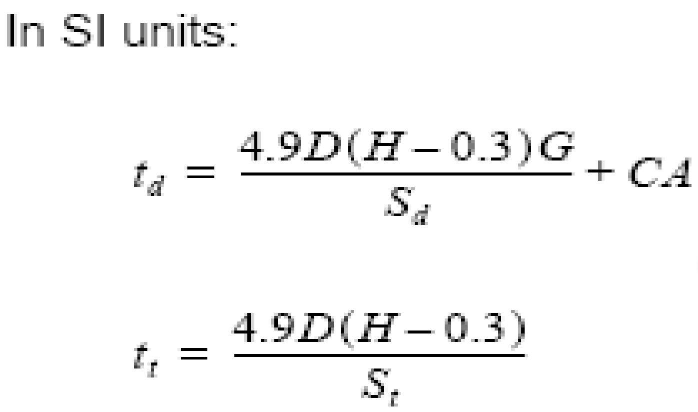
III) $Sc = R\rho g h / t$ Or $t = D \times G \times 9.8 \times H / S$

$t = 4.9\, D\, G\, H / S$

DESIGN (SHELL DESIGN)

- 5.6.3. Calculation of Thickness by the 1-Foot Method

  - This method shall not be used for tanks larger than 61 m (200 ft) in diameter.
  - The required minimum thickness of shell plates shall be the greater of the values computed by the following formulas:



In SI units:

$$t_d = \frac{4.9D(H-0.3)G}{S_d} + CA$$

$$t_t = \frac{4.9D(H-0.3)}{S_t}$$

$t = 4.9\,D\,G\,(H-0.3)\,/\,S$

Example

- Inside diameter: 34000 mm
- Tank Height : 8000 mm
- Corrosion allowance : 1.5 mm for all plates
- Material : A 283 C
- Service : Water, Density : 1000 Kg/m3
- Design pressure: Atm.
- Design liquid level: 6600 mm
- Plate width : 1500 mm + 500 mm for last course

Example

- Course # 1 :

t1 = Max. (td , tt)

Td = 4.9x 34m x (6.6 – 0.3) x 1 / 137 = 7.66 mm + 1.5 mm = 9.16 mm

Tt = 4.9 x 34 m x (6.6 – 0.3) /154 = 6.82 mm

So, The minimum required thickness of first shell course is Max (9.16 , 6.82)=9.16 and the selected thickness for first course is 10 mm

Example

- Course # 2 :

t2 = Max. (td , tt)

Td = 4.9x 34m x (6.6 –1.5- 0.3) x 1 / 137 = 5.83 mm + 1.5 mm = 7.33 mm

Tt = 4.9 x 34 m x (6.6 – 1.5- 0.3) /154 = 5.19 mm

So, The minimum required thickness of 2th shell course is Max (7.33 , 5.19)=7.33 and the selected thickness for 2th course is 8 mm

Example

THICKNESS OF SHELL PALTES

Course No	course height(mm)	H(mm)	St (Mpa)	Sd(Mpa)	T test(mm)	T design (mm)	Tactual (mm)	Material
1	1500	6600	154	137	6.82	9.1612	10	A 283 C or equivalent
2	1500	5100	154	137	5.19	7.3371	8	A 283 C or equivalent
3	1500	3600	154	137	3.57	5.5130	6	A 283 C or equivalent
4	1500	2100	154	137	1.95	3.6889	6	A 283 C or equivalent
5	1500	600	154	137	0.32	1.8648	6	A 283 C or equivalent
6	500	-900	154	137	-1.30	1.1566	6	A 283 C or equivalent

$$t_d = \frac{4.9D(H-0.3)G}{S_d} + CA \qquad t_t = \frac{4.9D(H-0.3)}{S_t}$$

1) Question!

- Inside diameter: 30000 mm
- Tank Height : 16000 mm
- Corrosion allowance : 3 mm for all plates
- Material :

✓ Course 1,2 : A 516 70

✓ Other courses: A 283 C

- Service : Gasoil, Density : 900 Kg/m3
- Design pressure: Atm.
- Design liquid level: 14200 mm
- Plate width : 2000 mm
- Minimum Amb. Temerature : -15 °C

☐ Shell and Bottom Plate thickness ?

☐ Impact test?

2) Question!

- Inside diameter: 8000 mm
- Tank Height : 6000 mm
- Corrosion allowance : 3 mm for all plates
- Material :A 283 C
- Service : Oil, Density : 850 Kg/m3
- Design pressure: Atm.
- Design liquid level: 5100 mm
- Plate width : 1800 mm
- Minimum Amb. Temerature : -12 °C

☐ Shell and Bottom Plate thickness ?

☐ Impact test?

شرکت طراحی مهندسی صنایع انرژی FIED	برگ محاسبات	صفحه : 1 از: تغییر : ☐☐☐☐☐

نام پروژه : API 650 MANUAL CALC.	شماره پروژه :	تهیه کننده : حسین [illegible] تاریخ :
موضوع : روش VARIABLE DESIGN POINT		کنترل کننده : تاریخ :

① زمانی از روش Variable design point استفاده می‌شود که نسبت $L/H \geq \frac{1000}{6}$ باشد. در کار ما روش 1-foot method را مشخص نکرده باشد.

$$L = \sqrt{500Dt}$$

* حداقل ضخامت های ذکر شده در بند 5.6.1.1 بایستی رعایت شود

② * مقدار اولیه t_{pd} برای حالت design و t_{pt} برای حالت hydrotest بایستی براساس فرمول‌ها روش one-foot method محاسبه گردد.

$$t_d = \frac{4.9D(H-0.3)G}{S_d} + CA$$

$$t_t = \frac{4.9D(H-0.3)}{S_t}$$

③ اولین کورس براساس روابط زیر برای محاسبه گردد

$$t_{1d} = \left(1.06 - \frac{0.0696D}{H}\sqrt{\frac{HG}{S_d}}\right)\left(\frac{4.9HDG}{S_d}\right) + CA$$

$$t_{1t} = \left(1.06 - \frac{0.696D}{H}\sqrt{\frac{H}{S_t}}\right)\left(\frac{4.9HD}{S_t}\right)$$

FOR DESIGN CONDITION t_{1d} NEED NOT BE GREATER THAN t_{pd}

FOR TEST CONDITION t_{1t} NEED NOT BE GREATER THAN t_{pt}

④ برای محاسبه ضخامت کورس بعدی بایستی ratio زیر برای کورس پایینی محاسبه گردد

$$\frac{h_1}{\sqrt{rt_1}}$$

$h_1 =$ ارتفاع کورس اول

$r =$ شعاع تانک

شرکت طراحی مهندسی صنایع انرژی FIED	**برگ محاسبات**	صفحه: 2 از: تغییر:

نام پروژه: API 650 - MANUAL CALC	شماره پروژه:	تهیه کننده: حسین هادی تاریخ:
موضوع: روش VARIABLE DESIGN POINT		کنترل کننده: تاریخ:

t_1 = ضخامت خورده شده کورس اول که برای محاسبه ضخامت t_2 نیاز است (برای TEST و DESIGN)

اگر Ratio ی محاسبه شده $1.375 \geq$ ⇐ $t_2 = t_1$

اگر ratio $2.625 \leq$ ⇐ $t_2 = t_{2a}$

اگر $1.375 < \text{ratio} < 2.625$ ⇐ $t_2 = t_{2a} + (t_1 - t_{2a})\left[2.1 - \frac{h_1}{1.25(rt_1)^{0.5}}\right]$

t_{2a} = ضخامت خورده شده کورس دوم، مطابق با 5.6.4.6 و 5.6.4.7

⑤ برای محاسبه کورس های بالاتر، مقدار اولیه t_u که برابر با one foot method است می بایست در نظر گرفته شود. و مقدار x لازم برای طراحی می بایست از کمترین مقادیر زیر انتخاب گردد.

$x_1 = 0.61(rt_u)^{0.5} + 320\,CH$

$x_2 = 1000\,CH$

$x_3 = 1.22(rt_u)^{0.5}$

$\Rightarrow C = [K^{0.5}(K-1)]/(1+K^{1.5})$

$\Rightarrow K = t_L/t_u$

$\Rightarrow$ t_L = CORRODED THICKNESS OF THE LOWER SHELL COURSE

$\Rightarrow$ H = DESIGN LIQUID LEVEL

برای ضخامت کورس های بالاتر داریم:

$$t_x = \text{Max.}\begin{cases} t_{dx} = \dfrac{4.9D\left(H - \frac{x}{1000}\right)G}{S_d} + CA \\ t_{tx} = \dfrac{4.9D\,(H - x/1000)}{S_t} \end{cases}$$

مراحل بعد می بایست آنقدر ادامه پیدا کند تا t_x Converge گردد.

BOTTOM AND ANNULAR PLATE

Bottom Plates

- ✓ 5.4. Bottom Plates

5.4.1 All bottom plates shall have a corroded thickness of not less than 6 mm . All rectangular and sketch plates shall have a nominal width of not less than 1800 mm.

5.4.2 Bottom plates of sufficient size shall be ordered so that, when trimmed, at least a 50 mm (2 in.) width will project outside the shell or meet requirements given in 5.1.5.7 d whichever is greater.

5.4.4 Unless otherwise specified on the Data Sheet, Line 12, tank bottoms requiring sloping shall have a minimum slope of 1:120 upwards toward center of the tank.

DESIGN (ANNULAR BOTTOM PLATES)

- ✓ 5.5. ANNULAR BOTTOM PLATES

- 5.5.1. When the bottom shell course is designed using the allowable stress for materials in Group IV, IVA, V, or VI, butt-welded annular bottom plates shall be used (see 5.1.5.6). When the bottom shell course is of a material in Group IV, IVA, V, or VI and the maximum product stress (see 5.6.2.1) for the first shell course is less than or equal to 160 MPa (23,200 lbf/in.2) or the maximum hydrostatic test stress (see 5.6.2.2) for the first shell course is less than or equal to 171 MPa (24,900 lbf/in.2), lap-welded bottom plates (see 5.1.5.4) may be used in lieu of butt-welded annular bottom plates.

- 5.1.5.6 Bottom annular-plate radial joints shall be butt-welded in accordance with 5.1.5.5 and shall have complete penetration and complete fusion. The backing strip, if used, shall be compatible for welding the annular plates together

DESIGN (ANNULAR BOTTOM PLATES)

- 5.5.4 The ring of annular plates shall have a circular outside circumference, but may have a regular polygonal shape inside the tank shell, with the number of sides equal to the number of annular plates. These pieces shall be welded in accordance with 5.1.5.6 and 5.1.5.7, Item b.
- 5.5.5 In lieu of annular plates, the entire bottom may be butt-welded provided that the requirements for annular plate thickness, welding, materials, and inspection are met for the annular distance specified in 5.5.2.
- 5.5.3. The thickness of the annular bottom plates shall not be less than the greater thickness determined using Table 5.1a and Table 5.1b for product design (plus any specified corrosion allowance) or for hydrostatic test design. Table 5.1a and Table 5.1b are applicable for effective product height of $H \times G \leq 23$ m (75 ft). Beyond this height an elastic analysis must be made to determine the annular plate thickness.

DESIGN (ANNULAR BOTTOM PLATES)

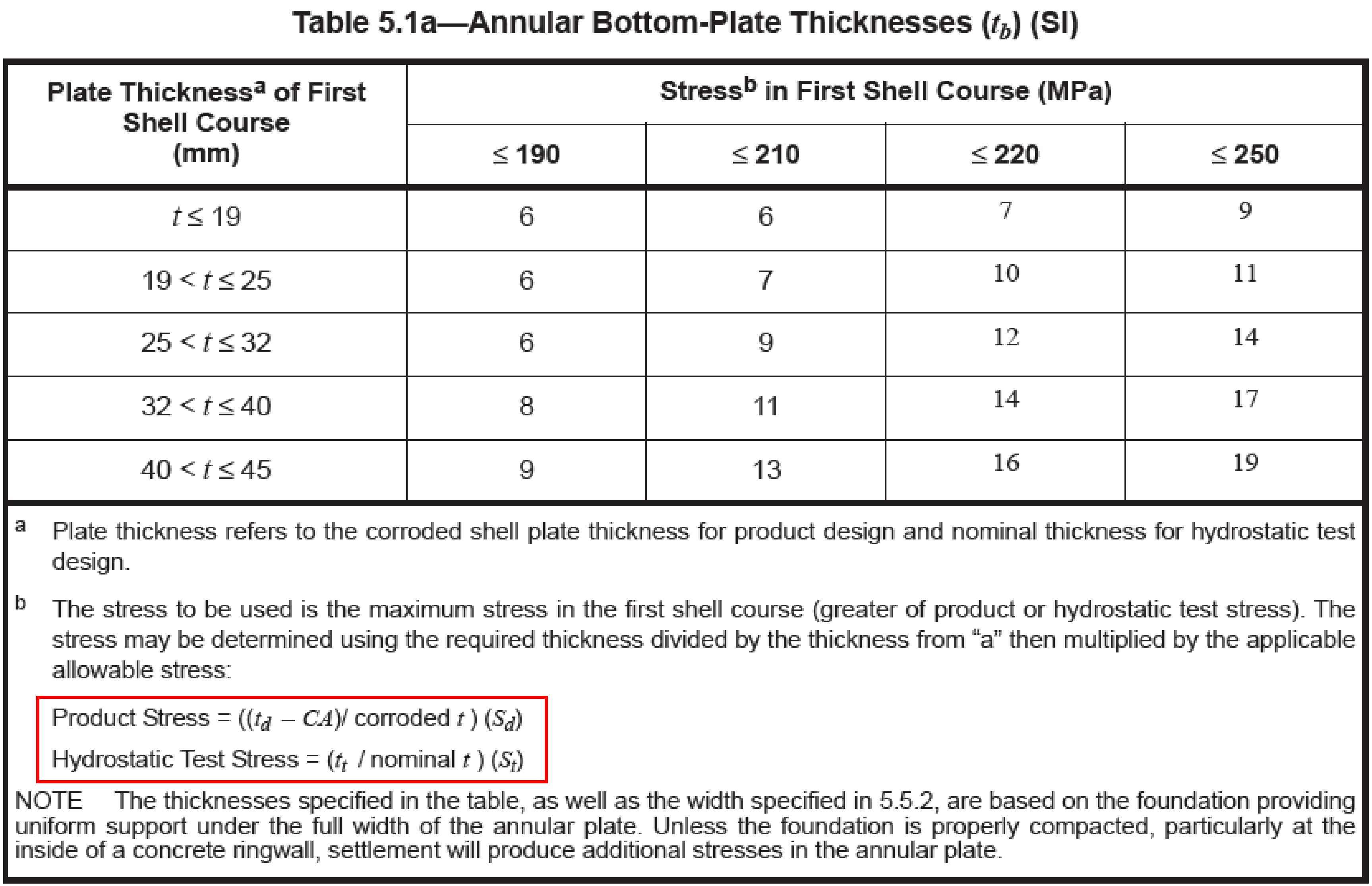


Table 5.1a—Annular Bottom-Plate Thicknesses (t_b) (SI)

Plate Thickness[a] of First Shell Course (mm)	Stress[b] in First Shell Course (MPa)			
	≤ 190	≤ 210	≤ 220	≤ 250
$t \leq 19$	6	6	7	9
$19 < t \leq 25$	6	7	10	11
$25 < t \leq 32$	6	9	12	14
$32 < t \leq 40$	8	11	14	17
$40 < t \leq 45$	9	13	16	19

[a] Plate thickness refers to the corroded shell plate thickness for product design and nominal thickness for hydrostatic test design.

[b] The stress to be used is the maximum stress in the first shell course (greater of product or hydrostatic test stress). The stress may be determined using the required thickness divided by the thickness from "a" then multiplied by the applicable allowable stress:

Product Stress = $((t_d - CA)/\text{ corroded } t)\ (S_d)$

Hydrostatic Test Stress = $(t_t\ /\text{ nominal } t)\ (S_t)$

NOTE The thicknesses specified in the table, as well as the width specified in 5.5.2, are based on the foundation providing uniform support under the full width of the annular plate. Unless the foundation is properly compacted, particularly at the inside of a concrete ringwall, settlement will produce additional stresses in the annular plate.

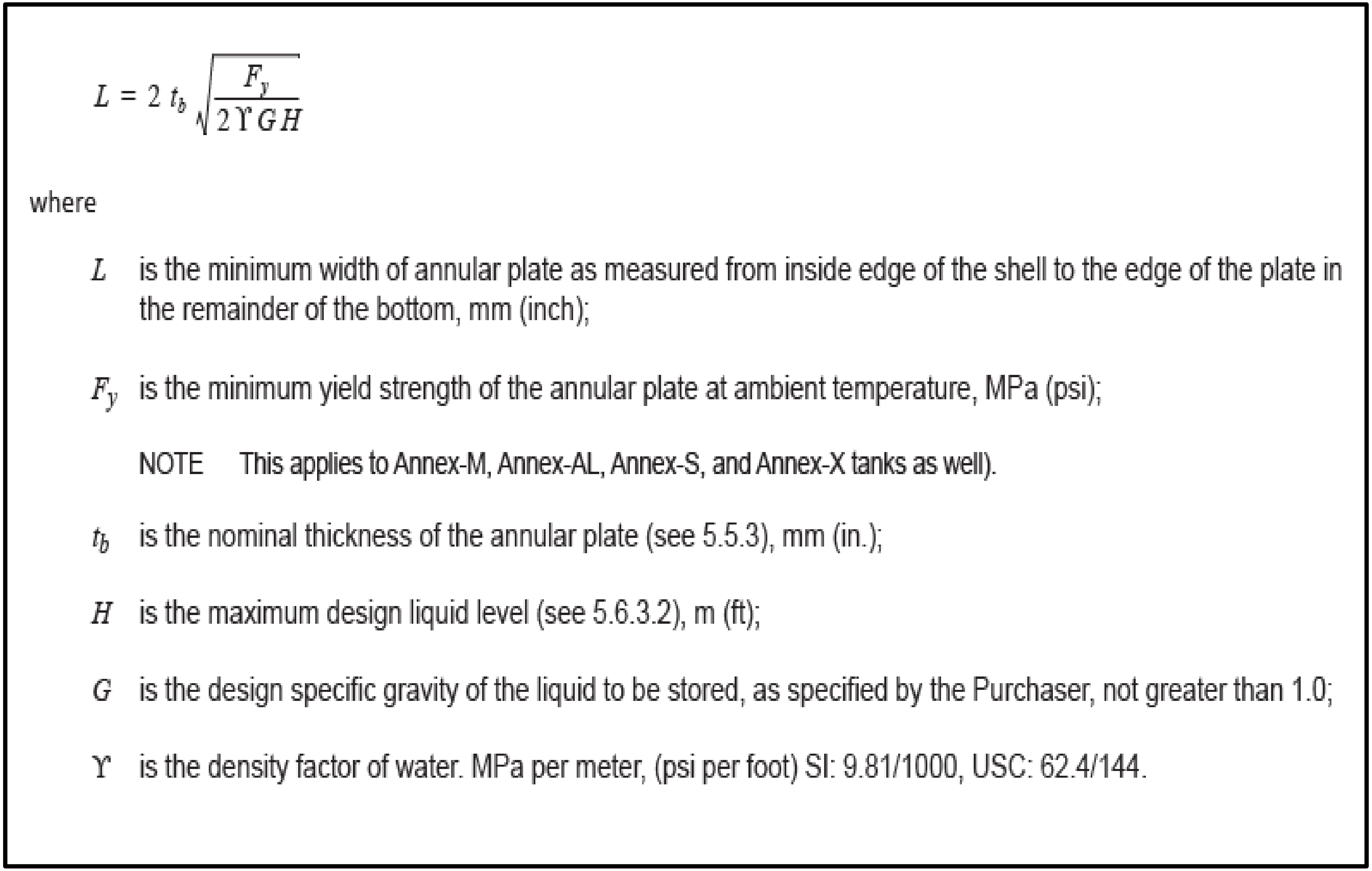
DESIGN (ANNULAR BOTTOM PLATES)

- 5.5.2. Annular bottom plates shall have a radial width that provides at least 600 mm (24 in.) between the inside of the shell and any lap-welded joint in the remainder of the bottom. Annular bottom plate projection outside the shell shall meet the requirements of 5.4.2. A greater radial width of annular plate is required when calculated as follows:

- Minimum Width of Annular plate :

overlap length(min. 5xtb)+tshell+50 mm + max. (L ; 600)

DESIGN (ANNULAR BOTTOM PLATES)



$$L = 2\,t_b \sqrt{\frac{F_y}{2\,\Upsilon\,G\,H}}$$

where

L is the minimum width of annular plate as measured from inside edge of the shell to the edge of the plate in the remainder of the bottom, mm (inch);

F_y is the minimum yield strength of the annular plate at ambient temperature, MPa (psi);

NOTE This applies to Annex-M, Annex-AL, Annex-S, and Annex-X tanks as well).

t_b is the nominal thickness of the annular plate (see 5.5.3), mm (in.);

H is the maximum design liquid level (see 5.6.3.2), m (ft);

G is the design specific gravity of the liquid to be stored, as specified by the Purchaser, not greater than 1.0;

Υ is the density factor of water. MPa per meter, (psi per foot) SI: 9.81/1000, USC: 62.4/144.

WIND GIRDER

Top Wind Girder

5.9.6 TOP WIND GIRDER

- 5.9.6.1. The required minimum section modulus of the stiffening ring shall be determined by the following equation

$$Z = \frac{D^2 H_2}{17} \left(\frac{V}{190} \right)^2$$

- Z is the required minimum section modulus, in cm3;
- D is the nominal tank diameter (for tanks in excess of 61 m diameter, the diameter shall be considered to be 61 m when determining the section modulus), in meters (m);
- H2 is the height of the tank shell, in meters, including any freeboard provided above the maximum filling height as a guide for a floating roof;
- V is the design wind speed (3-sec gust), in km/h (see 5.2.1[k]).

Top Wind Girder

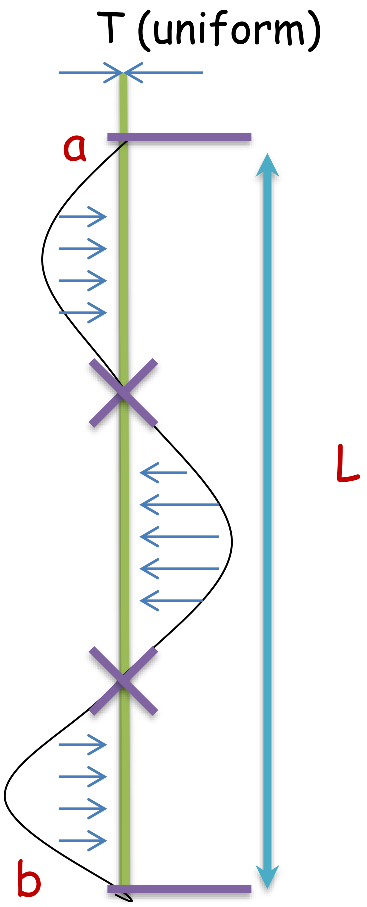
- 5.9.6.2 For tanks larger than 61 m (200 ft) in diameter, an additional check for the minimum required moment of inertia for the top-stiffening ring shall be performed. The required minimum moment of inertia of the stiffening ring shall be determined by the following equations:

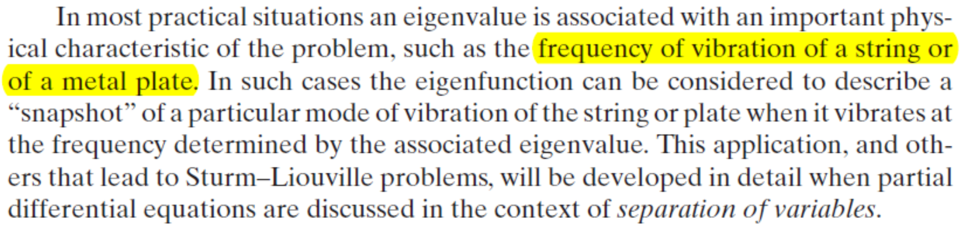
$$I = 3583 \times H_2 \times D^3 \times (V/190)^2/E$$

- I is the required minimum moment of inertia (cm4);
- D is the nominal diameter of the tank, in meters (m);
- H2 is the height of the tank shell (m), including any freeboard provided above the maximum filling height as a guide for a floating roof;
- E is the modulus of elasticity (MPa) at maximum design temperature;
- V is the design wind speed (3-sec gust) (km/h) (see 5.2.1[k]).

Intermediate Wind Girder

❖ Sturm-Liouville Equations (Singular Sturm-Liouville problems)

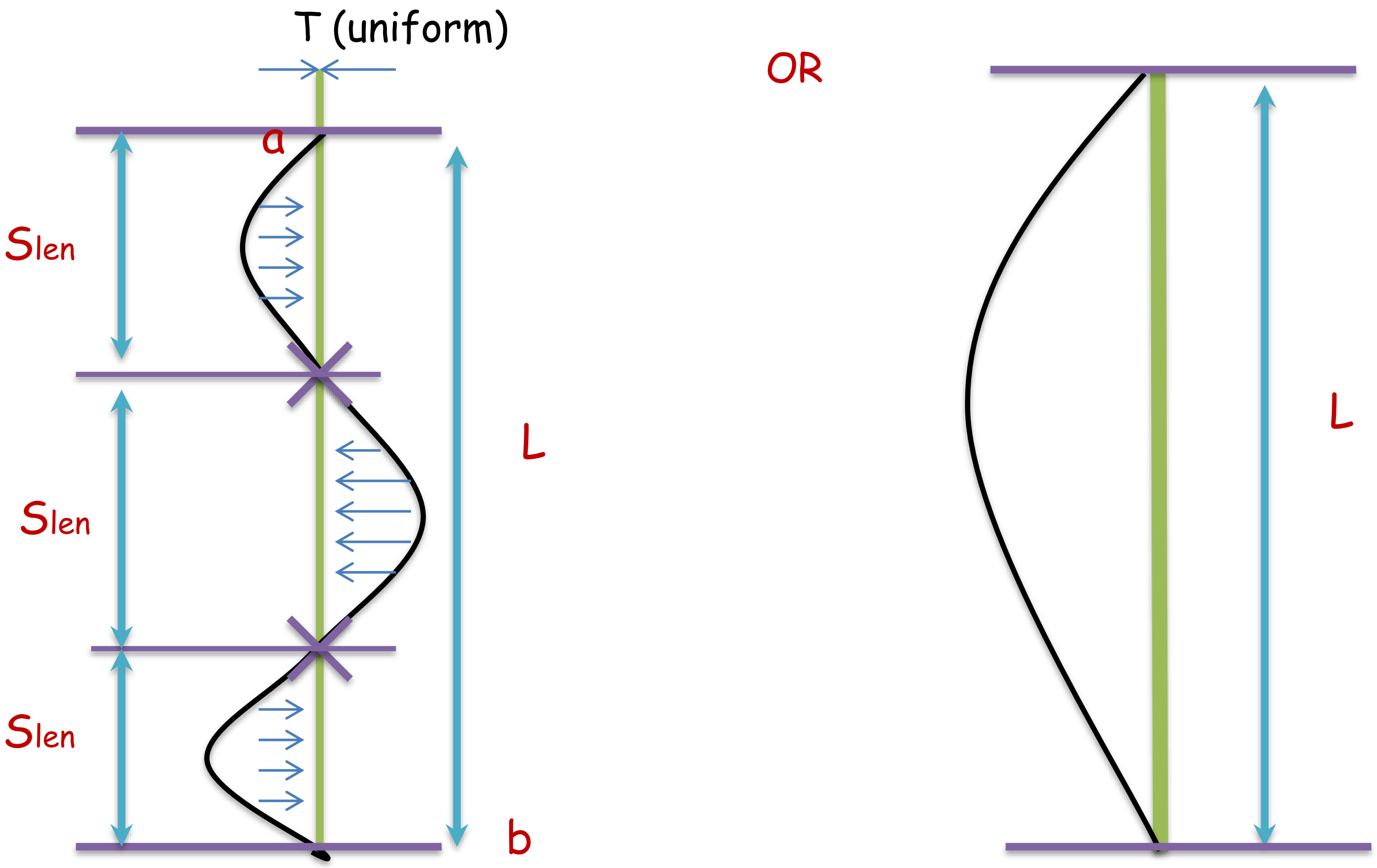



In most practical situations an eigenvalue is associated with an important physical characteristic of the problem, such as the frequency of vibration of a string or of a metal plate. In such cases the eigenfunction can be considered to describe a "snapshot" of a particular mode of vibration of the string or plate when it vibrates at the frequency determined by the associated eigenvalue. This application, and others that lead to Sturm–Liouville problems, will be developed in detail when partial differential equations are discussed in the context of *separation of variables*.

Ref. : Advanced engineering mathematics by Alan Jeffrey (University of Newcastle-upon-Tyne)

Intermediate Wind Girder

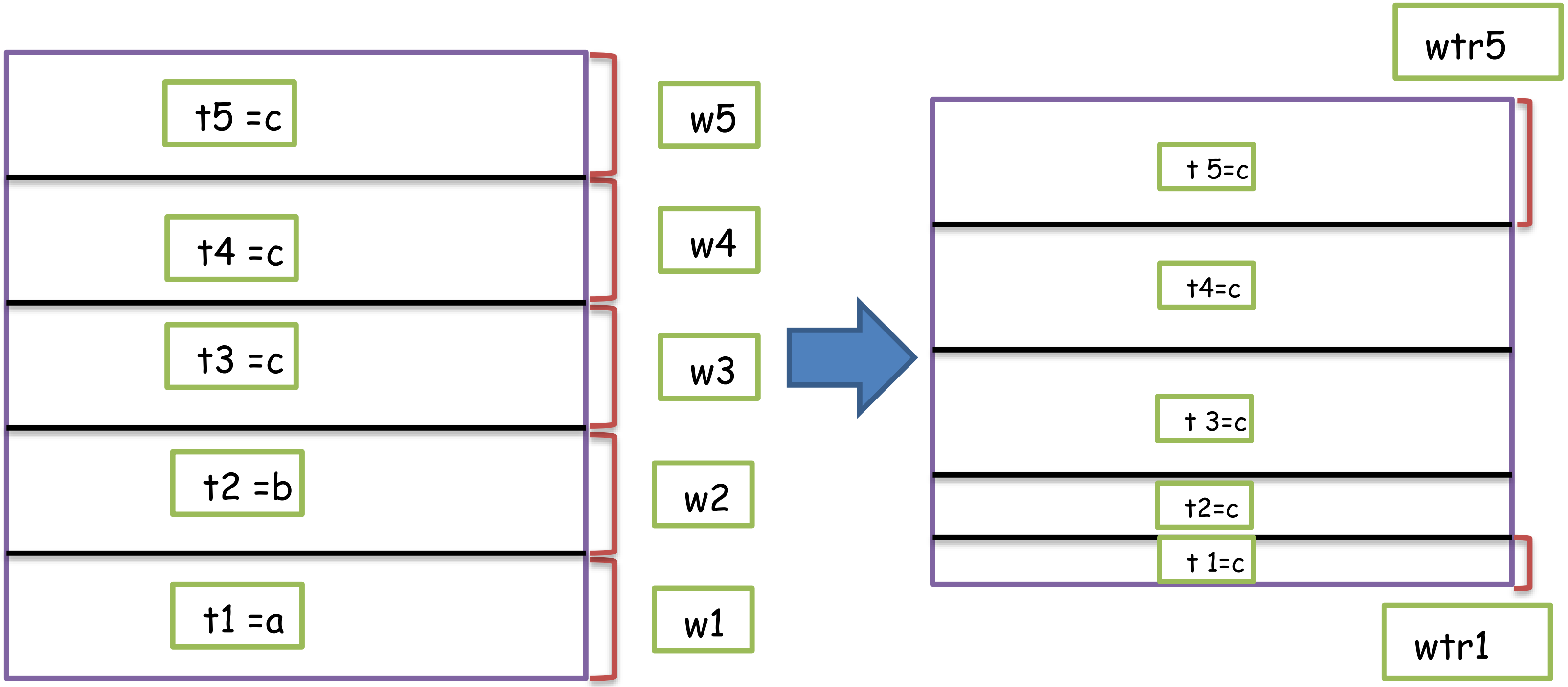
- Cylinder of pressure vessel



- See TABLE 8.1 Shear, moment, slope, andd eflection formulas for elastic straight beams

Intermediate Wind Girder

- If ∑Wtr > H1, wind girder is required



a > b > c

Intermediate Wind Girder

- The maximum height of the unstiffened shell shall be calculated as follows:

$$H_1 = 9.47t\sqrt{\left(\frac{t}{D}\right)^3}\left(\frac{190}{V}\right)^2$$

- H1 is the maximum height of the unstiffened shell, in meters;
- t is the nominal thickness, unless otherwise specified, of the thinnest shell course, in millimeters (see Note 1);
- D is the nominal tank diameter, in meters;

Intermediate Wind Girder

- Transposed width of each shell course having the top shell thickness:

$$W_{tr} = W\sqrt{\left(\frac{t_{\text{uniform}}}{t_{\text{actual}}}\right)^5}$$

- Wtr is the transposed width of each shell course, in millimeters (inches);
- W is the actual width of each shell course, in millimeters (inches); tuniform is the nominal thickness, unless otherwise specified, of the thinnest shell course, in millimeters (inches);
- tactual is the nominal thickness, unless otherwise specified, of the shell course for which the transposed width is being calculated, in millimeters (inches).

Example

- Inside diameter: 34000 mm
- Tank Height : 8000 mm
- Corrosion allowance : 1.5 mm for all plates
- Material : A 283 C
- Service : Water, Density : 1000 Kg/m3
- Design pressure: Atm.
- Design liquid level: 6600 mm
- Plate width : 1500 mm + 500 mm for last cour
- Minimum Amb. Temerature : -15 °C

❑ Anuular plate width and thickness?

❑ Wind girder calculation?

1) Question!

- Inside diameter: 30000 mm
- Tank Height : 16000 mm
- Corrosion allowance : 3 mm for all plates
- Material :

✓ Course 1,2 : A 516 70

✓ Other courses: A 283 C

- Service : Gasoil, Density : 900 Kg/m3
- Design pressure: Atm.
- Design liquid level: 14200 mm
- Plate width : 2000 mm
- Minimum Amb. Temerature : -15 °C

❑ Anuular plate width and thickness?

❑ Wind girder calculation?

2) Question!

- Inside diameter: 8000 mm
- Tank Height : 6000 mm
- Corrosion allowance : 3 mm for all plates
- Material :A 283 C
- Service : Oil, Density : 850 Kg/m3
- Design pressure: Atm.
- Design liquid level: 5100 mm
- Plate width : 1800 mm
- Minimum Amb. Temerature : -12 °C

☐ Anuular plate width and thickness?

☐ Wind girder calculation?

Intermediate Wind Girder

- Transposed width of each shell course having the top shell thickness:

$$W_{tr} = W\sqrt{\left(\frac{t_{\text{uniform}}}{t_{\text{actual}}}\right)^5}$$

- Wtr is the transposed width of each shell course, in millimeters (inches);
- W is the actual width of each shell course, in millimeters (inches); tuniform is the nominal thickness, unless otherwise specified, of the thinnest shell course, in millimeters (inches);
- tactual is the nominal thickness, unless otherwise specified, of the shell course for which the transposed width is being calculated, in millimeters (inches).

INTERMEDIATE WIND GIRDER

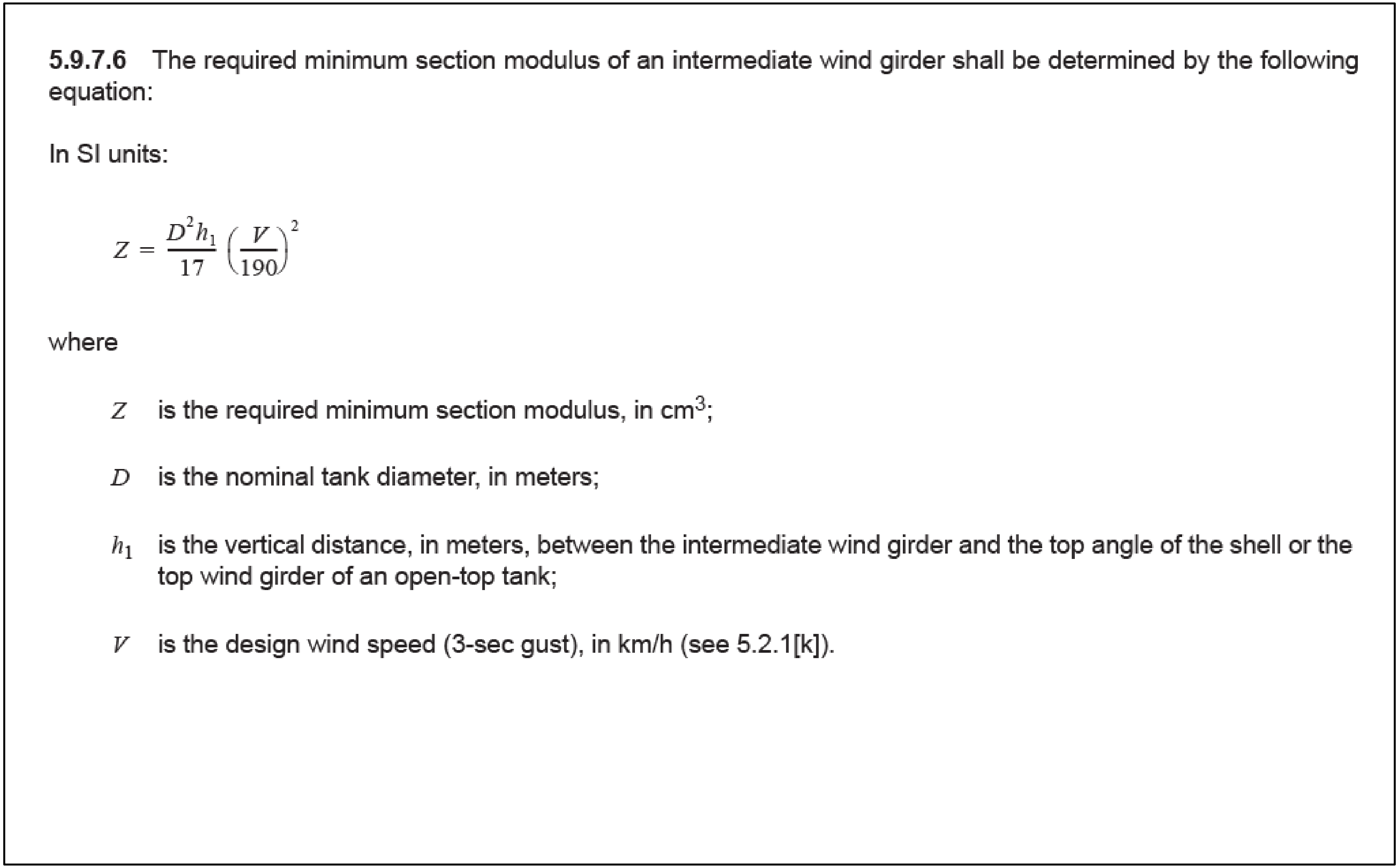
5.9.7.3 If the height of the transformed shell is greater than the maximum height H1, an intermediate wind girder is required.

5.9.7.3.1 For equal stability above and below the intermediate wind girder, the girder should be located at the midheight of the transformed shell. The location of the girder on the actual shell should be at the same course and same relative position as the location of the girder on the transformed shell, using the thickness relationship in 5.9.7.2.

INTERMEDIATE WIND GIRDER

- 5.9.7.3.2 Other locations for the girder may be used, provided the height of unstiffened shell on the transformed shell does not exceed H1 (see 5.9.7.5).
- 5.9.7.4 If half the height of the transformed shell exceeds the maximum height H1, a second intermediate girder shall be used to reduce the height of unstiffened shell to a height less than the maximum.
- 5.9.7.5 Intermediate wind girders shall not be attached to the shell within 150 mm (6 in.) of a horizontal joint of the shell. When the preliminary location of a girder is within 150 mm (6 in.) of a horizontal joint, the girder shall preferably be located 150 mm (6 in.) below the joint; however, the maximum unstiffened shell height shall not be exceeded.

INTERMEDIATE WIND GIRDER



5.9.7.6 The required minimum section modulus of an intermediate wind girder shall be determined by the following equation:

In SI units:

$$Z = \frac{D^2 h_1}{17}\left(\frac{V}{190}\right)^2$$

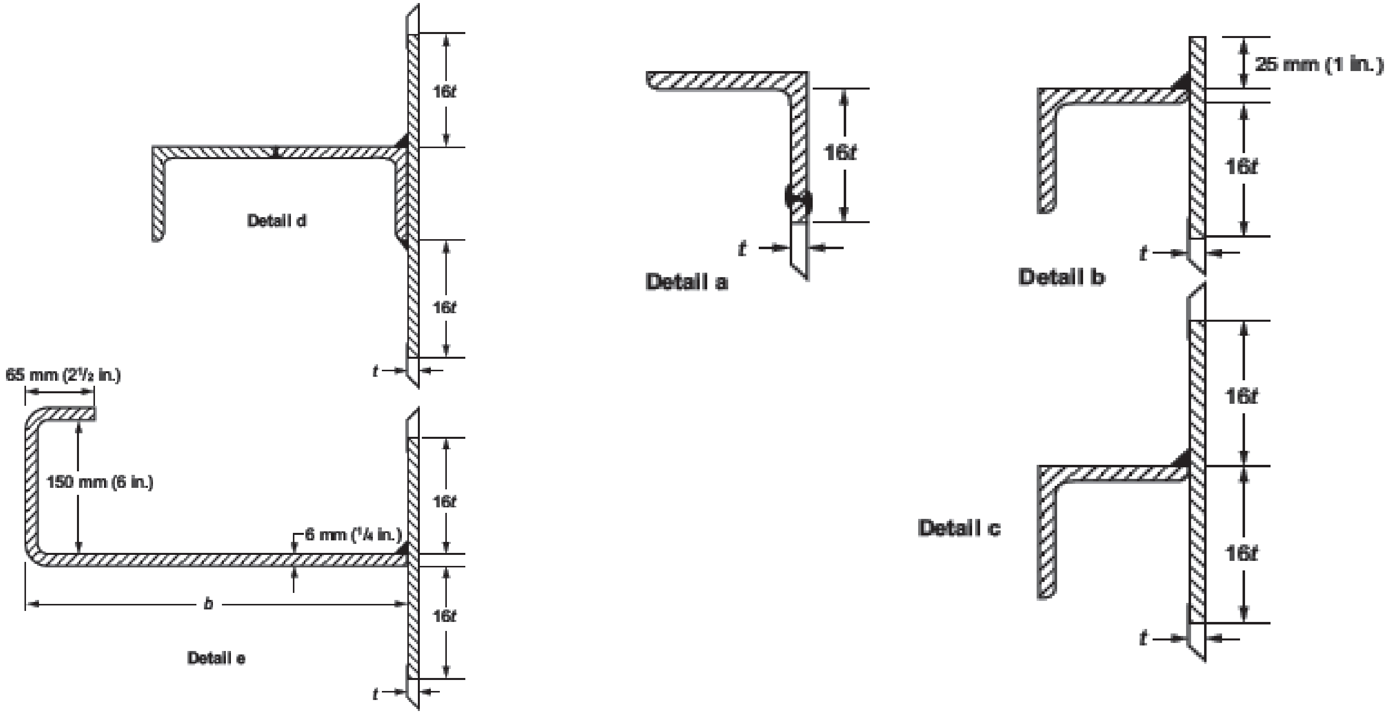
where

Z is the required minimum section modulus, in cm^3;

D is the nominal tank diameter, in meters;

h_1 is the vertical distance, in meters, between the intermediate wind girder and the top angle of the shell or the top wind girder of an open-top tank;

V is the design wind speed (3-sec gust), in km/h (see 5.2.1[k]).



Note: The section moduli given in Tables 5.20a and 5.20b for Details c and d are based on the longer leg being located horizontally (perpendicular to the shell) when angles with uneven legs are used.

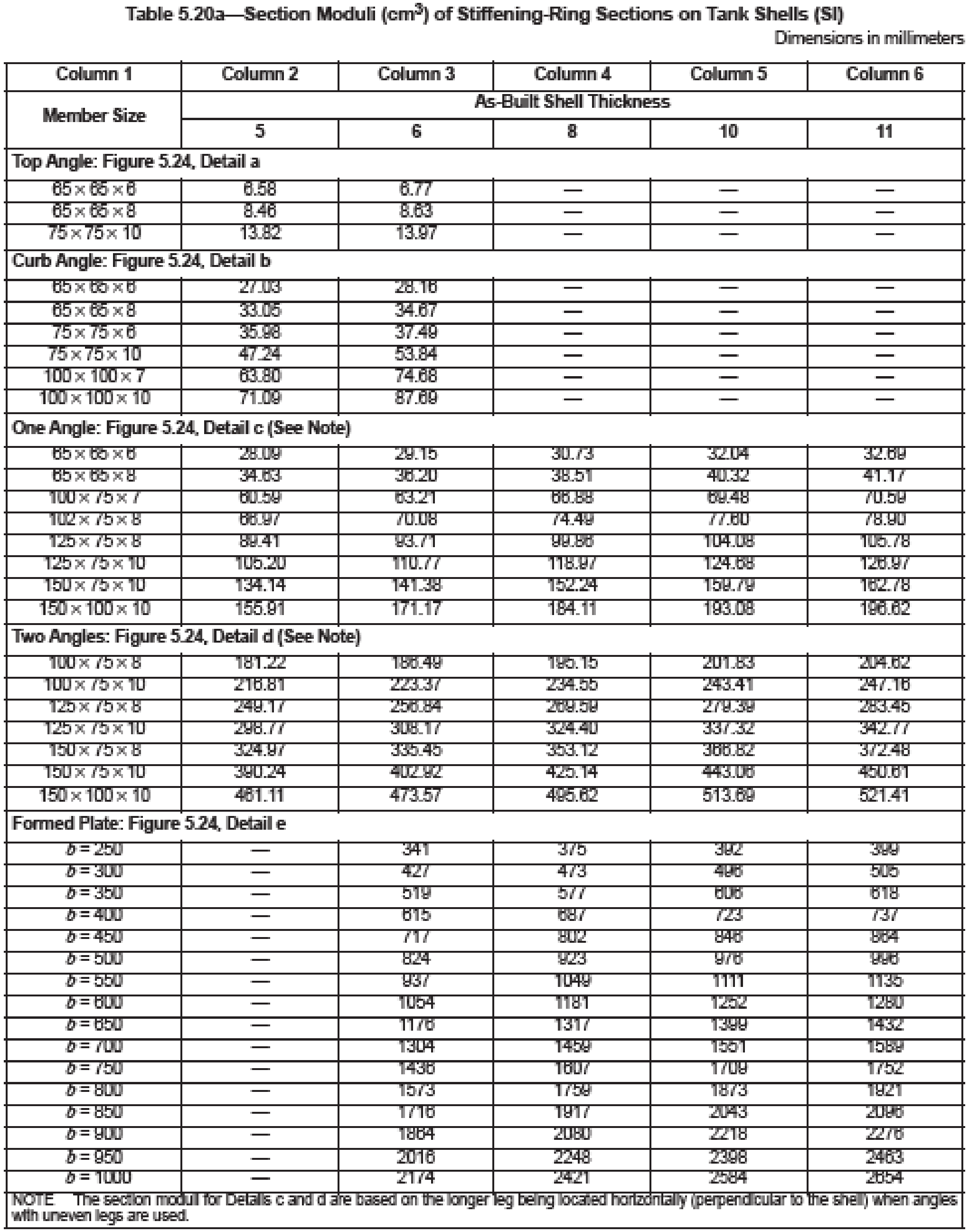


Table 5.20a—Section Moduli (cm^3) of Stiffening-Ring Sections on Tank Shells (SI)

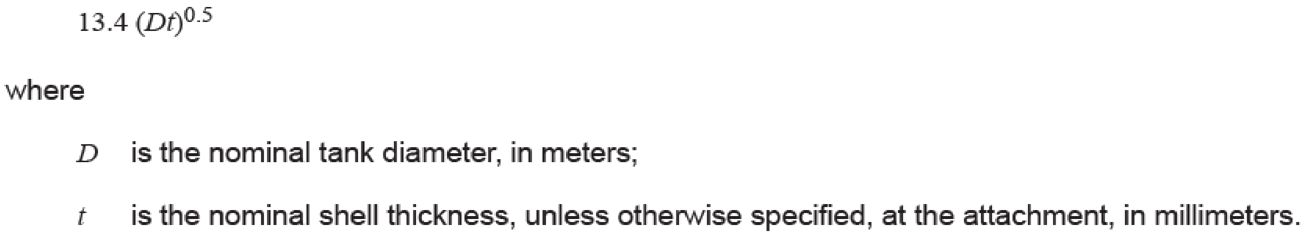
Dimensions in millimeters

Column 1	Column 2	Column 3	Column 4	Column 5	Column 6
Member Size	As-Built Shell Thickness				
	5	6	8	10	11
Top Angle: Figure 5.24, Detail a					
65 × 65 × 6	6.58	6.77	—	—	—
65 × 65 × 8	8.46	8.63	—	—	—
75 × 75 × 10	13.82	13.97	—	—	—
Curb Angle: Figure 5.24, Detail b					
65 × 65 × 6	27.03	28.16	—	—	—
65 × 65 × 8	33.05	34.67	—	—	—
75 × 75 × 6	35.98	37.49	—	—	—
75 × 75 × 10	47.24	53.84	—	—	—
100 × 100 × 7	63.80	74.68	—	—	—
100 × 100 × 10	71.09	87.69	—	—	—
One Angle: Figure 5.24, Detail c (See Note)					
65 × 65 × 6	28.09	29.15	30.73	32.04	32.69
65 × 65 × 8	34.63	36.20	38.51	40.32	41.17
100 × 75 × 7	60.59	63.21	66.88	69.48	70.59
102 × 75 × 8	66.97	70.08	74.49	77.60	78.90
125 × 75 × 8	89.41	93.71	99.86	104.08	105.78
125 × 75 × 10	105.20	110.77	118.97	124.68	126.97
150 × 75 × 10	134.14	141.38	152.24	159.79	162.78
150 × 100 × 10	155.91	171.17	184.11	193.08	196.62
Two Angles: Figure 5.24, Detail d (See Note)					
100 × 75 × 8	181.22	186.49	195.15	201.83	204.62
100 × 75 × 10	216.81	223.37	234.55	243.41	247.16
125 × 75 × 8	249.17	256.84	269.59	279.39	283.45
125 × 75 × 10	298.77	308.17	324.40	337.32	342.77
150 × 75 × 8	324.97	335.45	353.12	366.82	372.48
150 × 75 × 10	390.24	402.92	425.14	443.06	450.61
150 × 100 × 10	461.11	473.57	495.62	513.69	521.41
Formed Plate: Figure 5.24, Detail e					
$b = 250$	—	341	375	392	399
$b = 300$	—	427	473	496	505
$b = 350$	—	519	577	606	618
$b = 400$	—	615	687	723	737
$b = 450$	—	717	802	846	864
$b = 500$	—	824	923	976	996
$b = 550$	—	937	1049	1111	1135
$b = 600$	—	1054	1181	1252	1280
$b = 650$	—	1176	1317	1399	1432
$b = 700$	—	1304	1459	1551	1589
$b = 750$	—	1436	1607	1709	1752
$b = 800$	—	1573	1759	1873	1921
$b = 850$	—	1716	1917	2043	2096
$b = 900$	—	1864	2080	2218	2276
$b = 950$	—	2016	2248	2398	2463
$b = 1000$	—	2174	2421	2584	2654

NOTE The section moduli for Details c and d are based on the longer leg being located horizontally (perpendicular to the shell) when angles with uneven legs are used.

INTERMEDIATE WIND GIRDER

- 5.9.7.6.2 The section modulus of the intermediate wind girder shall be based on the properties of the attached members and may include a portion of the tank shell for a distance above and below the attachment to the shell, in mm (in.)



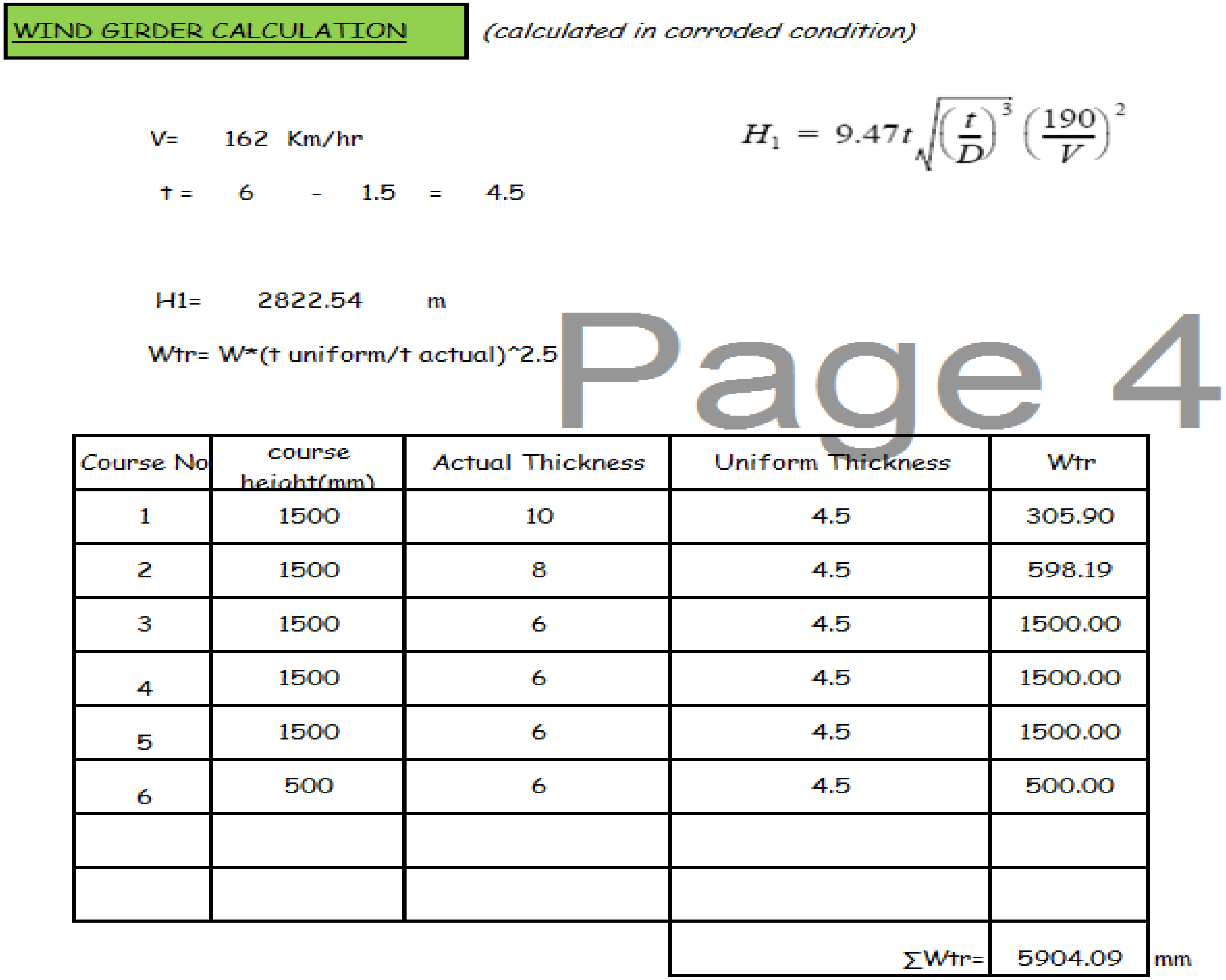
$13.4\,(Dt)^{0.5}$

where

D is the nominal tank diameter, in meters;

t is the nominal shell thickness, unless otherwise specified, at the attachment, in millimeters.

INTERMEDIATE WIND GIRDER



WIND GIRDER CALCULATION (calculated in corroded condition)

V= 162 Km/hr

t = 6 - 1.5 = 4.5

$$H_1 = 9.47t\sqrt{\left(\frac{t}{D}\right)^3}\left(\frac{190}{V}\right)^2$$

H1= 2822.54 m

Wtr= W*(t uniform/t actual)^2.5

Course No	course height(mm)	Actual Thickness	Uniform Thickness	Wtr
1	1500	10	4.5	305.90
2	1500	8	4.5	598.19
3	1500	6	4.5	1500.00
4	1500	6	4.5	1500.00
5	1500	6	4.5	1500.00
6	500	6	4.5	500.00
			∑Wtr=	5904.09 mm

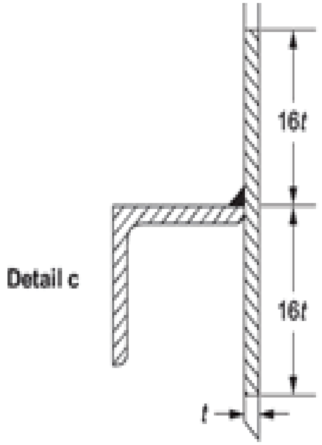
INTERMEDIATE WIND GIRDER

∑Wtr / H1 = 2.0918

The max. height of the unstiff. shell is less than tank height So, two intermediate Wind Girder is required at +4800 and +7000 elevations.

The required minimum section modulus of intermediate wind girders shall be determined by the following equation:

$$Z = \frac{D^2 H_1}{17} \left(\frac{V}{190} \right)^2 = 139.53 \text{ cm3}$$



Selected angle size for wind girder as per detail "C" of Figure 5-20 is: L150x75x10

Note: The section moduli for Detail C are based on the longer leg being located horizontally (perpendicular to the shell).

ROOF

ROOF

✓ 5.10 ROOFS

- Internal Floating roof
- External Floating roof
- A supported cone roof
- A Supported Dome roof
- A self-supporting cone roof
- A self-supporting dome roof
- A self-supporting umbrella roof
- Frangible roof

ROOF

- 5.10.2.2. Roof plates shall have a nominal thickness of not less than 5 mm (3/16 in.) or 7- gauge sheet. Increased thickness may be required for supported cone roofs (see 5.10.4.4). Any required corrosion allowance for the plates of self-supporting roofs shall be added to the calculated thickness unless otherwise specified by the Purchaser. Any corrosion allowance for the plates of supported roofs shall be added to the greater of the calculated thickness or the minimum thickness or [5 mm (3/16 in.) or 7-gauge sheet].

ROOF

- 5.10.4 Supported Cone Roofs

The slope of the roof shall be 1:16 (3.6°) or greater if specified by the Purchaser

- 5.10.5 Self-Supporting Cone Roofs

Self-supporting cone roofs shall conform to the following requirements:

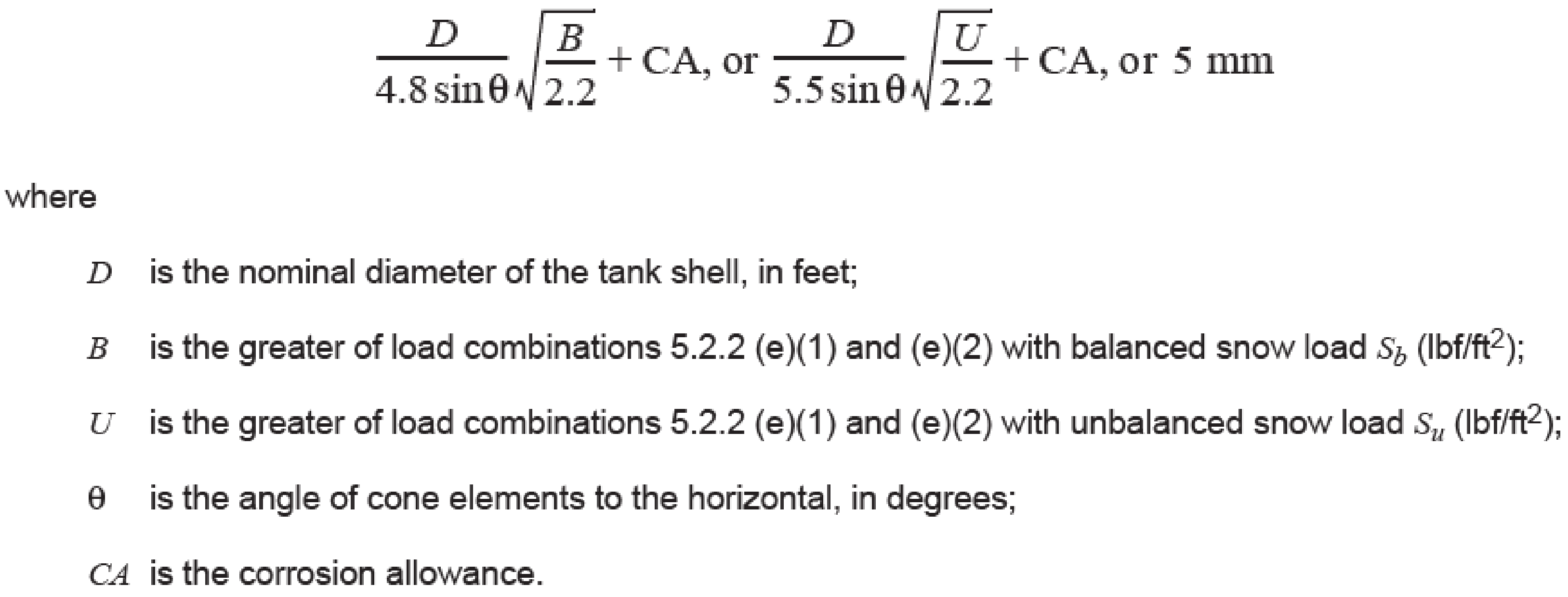
$\theta \leq 37$ degrees (slope = 9:12)

$\theta \geq 9.5$ degrees (slope = 2:12)

ROOF

- 5.10.5 Self-Supporting Cone Roofs

Nominal thickness shall not be less than the greatest of:



$$\frac{D}{4.8\sin\theta}\sqrt{\frac{B}{2.2}}+\text{CA, or }\frac{D}{5.5\sin\theta}\sqrt{\frac{U}{2.2}}+\text{CA, or 5 mm}$$

where

D is the nominal diameter of the tank shell, in feet;

B is the greater of load combinations 5.2.2 (e)(1) and (e)(2) with balanced snow load S_b (lbf/ft^2);

U is the greater of load combinations 5.2.2 (e)(1) and (e)(2) with unbalanced snow load S_u (lbf/ft^2);

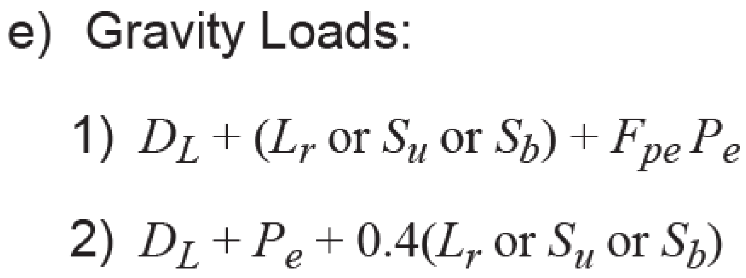
θ is the angle of cone elements to the horizontal, in degrees;

CA is the corrosion allowance.

Note : Corroded thickness shall not be more than 13 mm.

ROOF

- 5.2.2 Load Combinations



e) Gravity Loads:

1) $D_L + (L_r \text{ or } S_u \text{ or } S_b) + F_{pe} P_e$

2) $D_L + P_e + 0.4(L_r \text{ or } S_u \text{ or } S_b)$

- The external pressure combination factor (Fpe) is defined as the ratio of normal operating external pressure to design external pressure, with a minimum value of 0.4.

ROOF

- 5.10.6 Self-Supporting dome and umbrella Roofs

Nominal thickness shall not be less than the greatest of:

$$\frac{r_r}{2.4}\sqrt{\frac{B}{2.2}} + \text{CA, or } \frac{r_r}{2.7}\sqrt{\frac{U}{2.2}} + \text{CA, or 5 mm}$$

Minimum radius = $0.8D$ (unless otherwise specified by the Purchaser)

Maximum radius = $1.2D$

where

D is the nominal diameter of the tank shell, in meters;

B is the greater of load combinations 5.2.2 (e)(1) and (e)(2) with balanced snow load S_b (kPa);

U is the greater of load combinations 5.2.2 (e)(1) and (e)(2) with unbalanced snow load S_u (kPa);

r_r is the roof radius, in meters.

Note : Corroded thickness shall not be more than 13 mm.

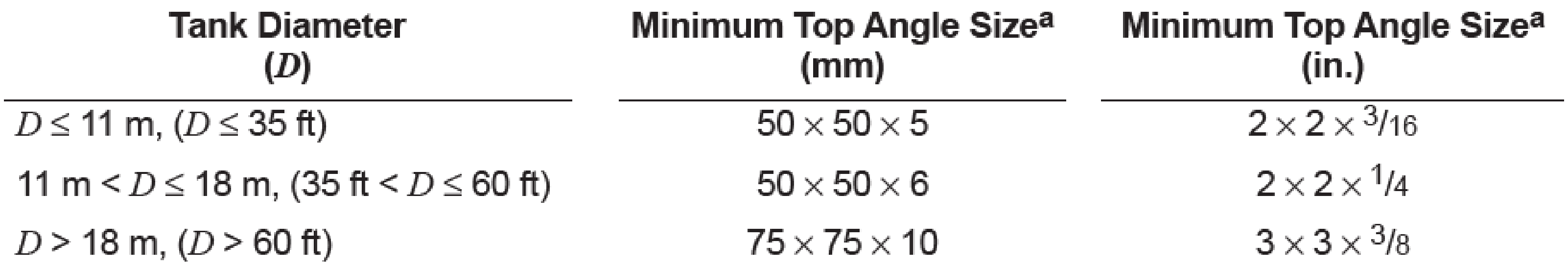
TOP ANGLE

Top Angle

- 5.1.5.9 Roof and Top-Angle Joints
- a) Roof plates shall, as a minimum, be welded on the top side with a continuous full-fillet weld on all seams. Buttwelds are also permitted.
- b) For frangible roofs, roof plates shall be attached to the top angle of a tank with a continuous fillet weld on the top side only, as specified in 5.10.2.6. For non-frangible roofs, alternate details are permitted.
- c) The top-angle sections, tension rings, and compression rings shall be joined by butt-welds having complete penetration and fusion. Joint efficiency factors need not be applied when conforming to the requirements of 5.10.5 and 5.10.6.
- d) At the option of the Manufacturer, for self-supporting roofs of the cone, dome, or umbrella type, the edges of the roof plates may be flanged horizontally to rest flat against the top angle to improve welding conditions.

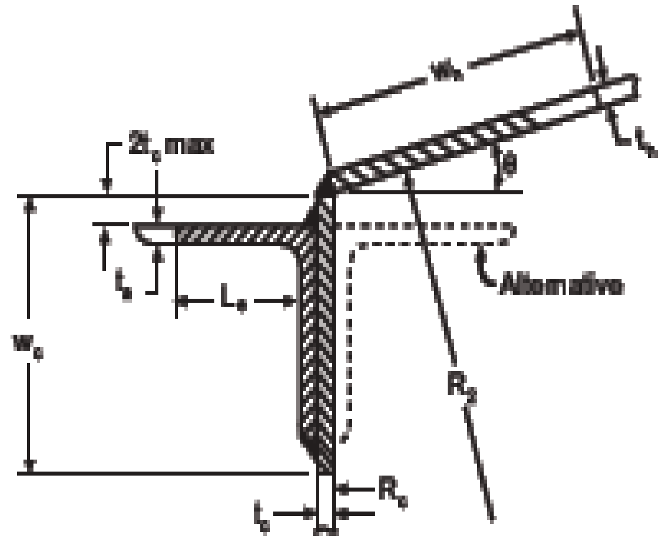
Top Angle

- e) Except as specified for open-top tanks in 5.9, for tanks with frangible joints per 5.10.2.6, for self-supporting roofs in 5.10.5, and 5.10.6, and for tanks with the flanged roof-to-shell detail described in Item f below, tank shells shall be supplied with top angles of not less than the following sizes:

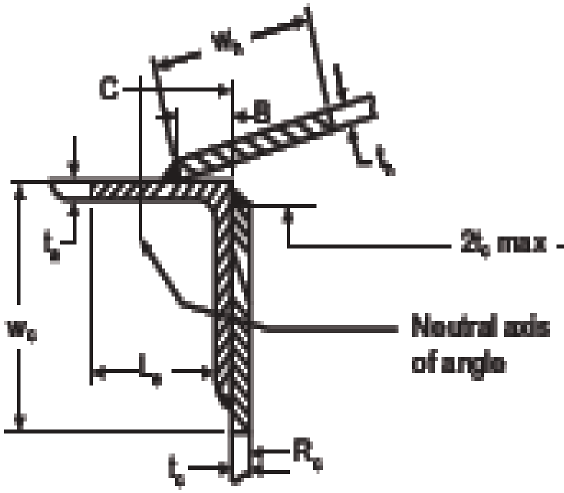


Tank Diameter (D)	Minimum Top Angle Size[a] (mm)	Minimum Top Angle Size[a] (in.)
$D \leq 11$ m, ($D \leq 35$ ft)	$50 \times 50 \times 5$	$2 \times 2 \times {}^{3}/_{16}$
11 m $< D \leq 18$ m, (35 ft $< D \leq 60$ ft)	$50 \times 50 \times 6$	$2 \times 2 \times {}^{1}/_{4}$
$D > 18$ m, ($D > 60$ ft)	$75 \times 75 \times 10$	$3 \times 3 \times {}^{3}/_{8}$

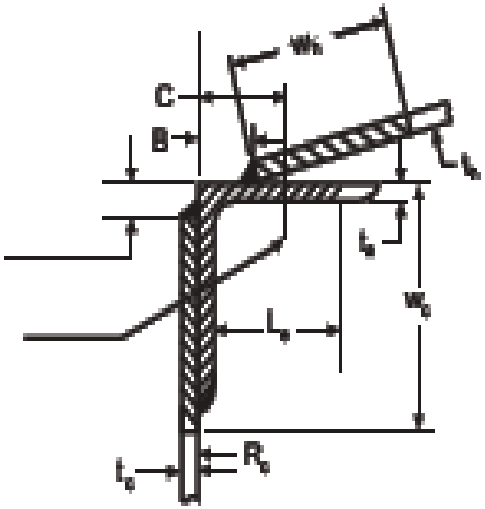
- Roof-to-shell connection details per Figure F.2 are permissible provided that the design effective area (crosshatched section) is greater than or equal to the design effective area provided by the minimum top angle size listed above.



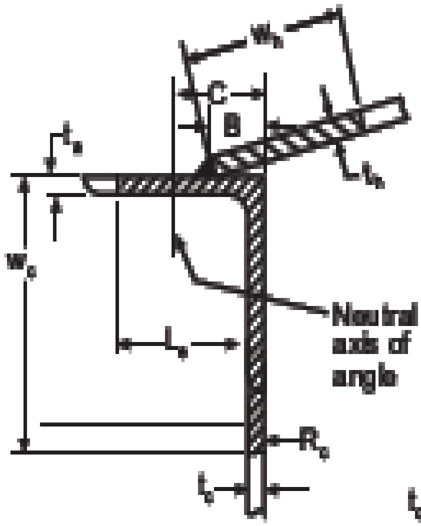
Detail a



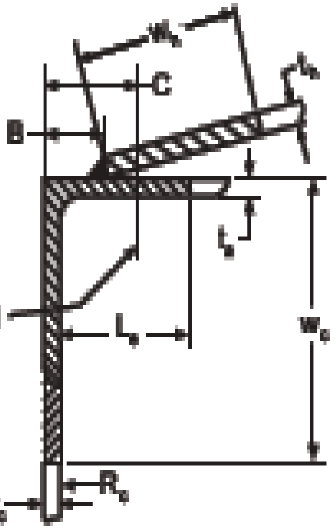
Detail b



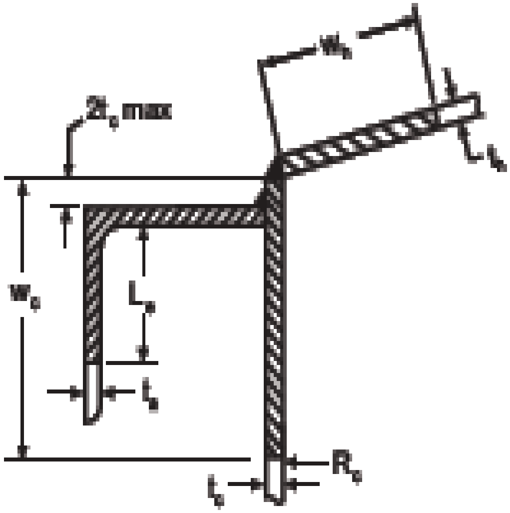
Detail c



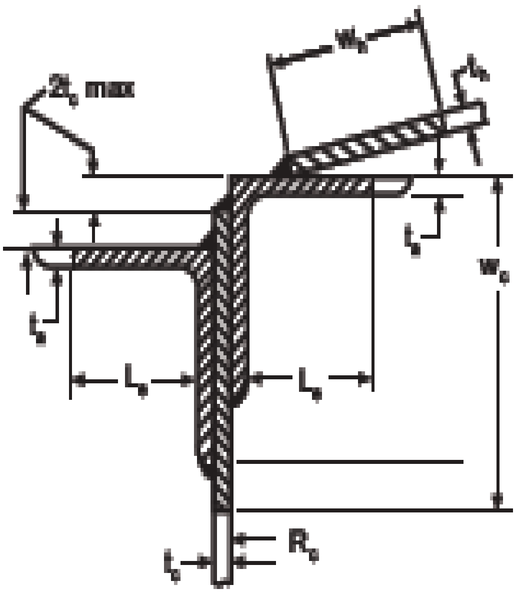
Detail d



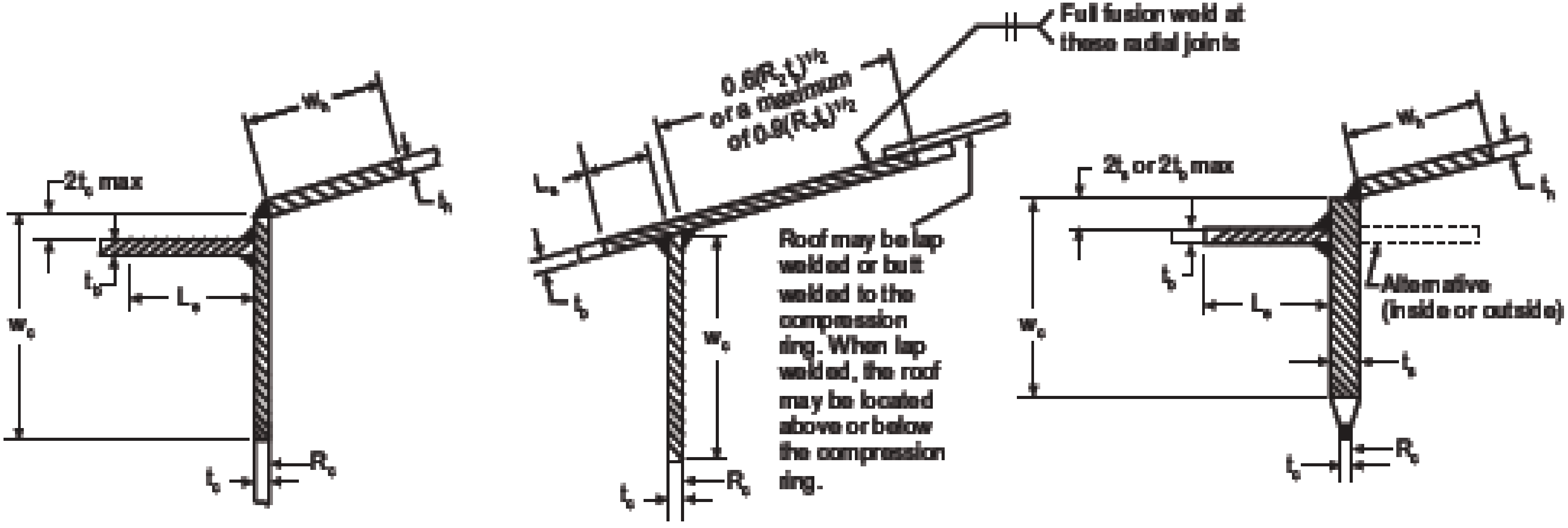
Detail e



Detail f



Detail g



Detail h

Detail i

Detail k

t_a = thickness of angle leg
t_b = thickness of bar
t_c = thickness of shell plate
t_h = thickness of roof plate
t_s = thickness of thickened plate in shell
t_i = t_a plus t_c (see note 4)
w_c = maximum width of participating shell $= 0.6\,(R_c\,t)^{1/2}$, where $t = t_a$, t_c, t_s, or t_i as applicable.

w_h = maximum width of participating roof $= 0.3(R_2\,t_h)^{1/2}$ or 300 mm (12 in.) whichever is less.
R_c = inside radius of tank shell
R_2 = length of the normal to the roof, measured from the vertical centerline of the tank $= R_c / (\sin\theta)$
θ = angle between roof and horizontal

NOTE 1 All dimensions and thicknesses are in millimeters (inches).

NOTE 2 Dimension B in details b, c, d, and e is: $0 \leq B \leq C$. C is the dimension to the neutral axis of the angle.

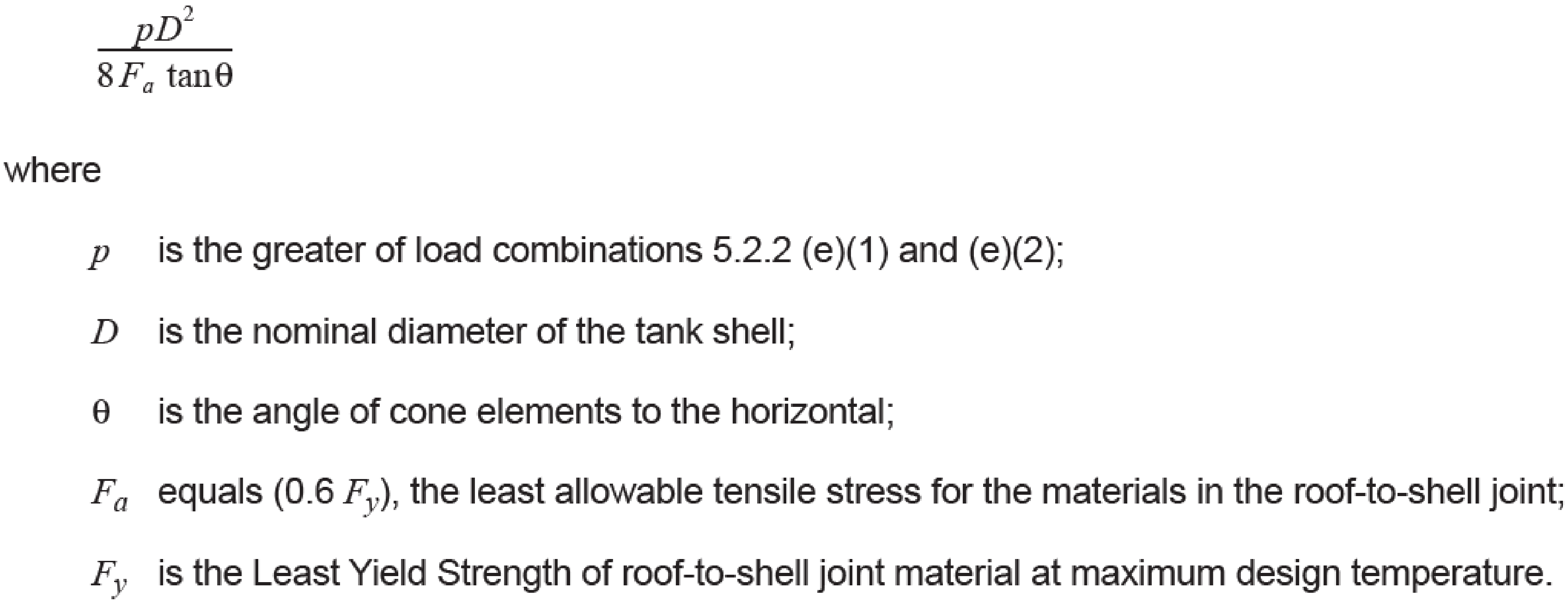
NOTE 3 The unstiffened length of the angle or bar, L_e, shall be limited to $250t/(F_y)^{1/2}$ mm [$3000t/(F_y)^{1/2}$ in.] where F_y is the minimum specified yield strength, MPa (lbf/in.^{2}) and $t = t_a$ or t_b, as applicable.

NOTE 4 Where members are lap welded onto the shell (refer to details a, b, c, and g), t_i may be used in w_c formula only for the extent of the overlap.

Figure F.2—Permissible Details of Compression Rings

Top Angle

- 5.10.2.2. The participating area at the roof-to-shell joint shall be determined using Figure F.2 and the nominal material thickness less any corrosion allowance shall equal or exceed the following:



$$\frac{pD^2}{8F_a \tan\theta}$$

where

p is the greater of load combinations 5.2.2 (e)(1) and (e)(2);

D is the nominal diameter of the tank shell;

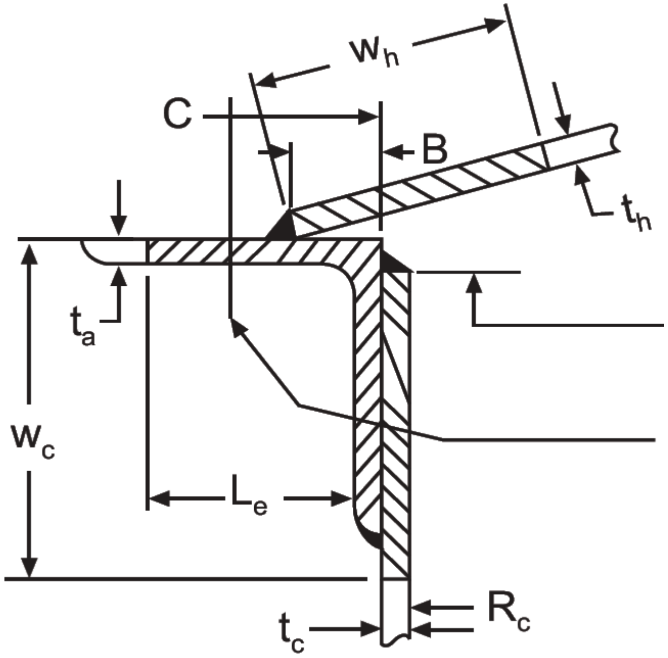
θ is the angle of cone elements to the horizontal;

F_a equals ($0.6\ F_y$), the least allowable tensile stress for the materials in the roof-to-shell joint;

F_y is the Least Yield Strength of roof-to-shell joint material at maximum design temperature.

Top Angle

A shell + A roof + A angle > A required



Detail b

Top Angle

- A shell = Wc x ts

- Wc = maximum width of participating shell = 0.6 (Rc t)^0.5

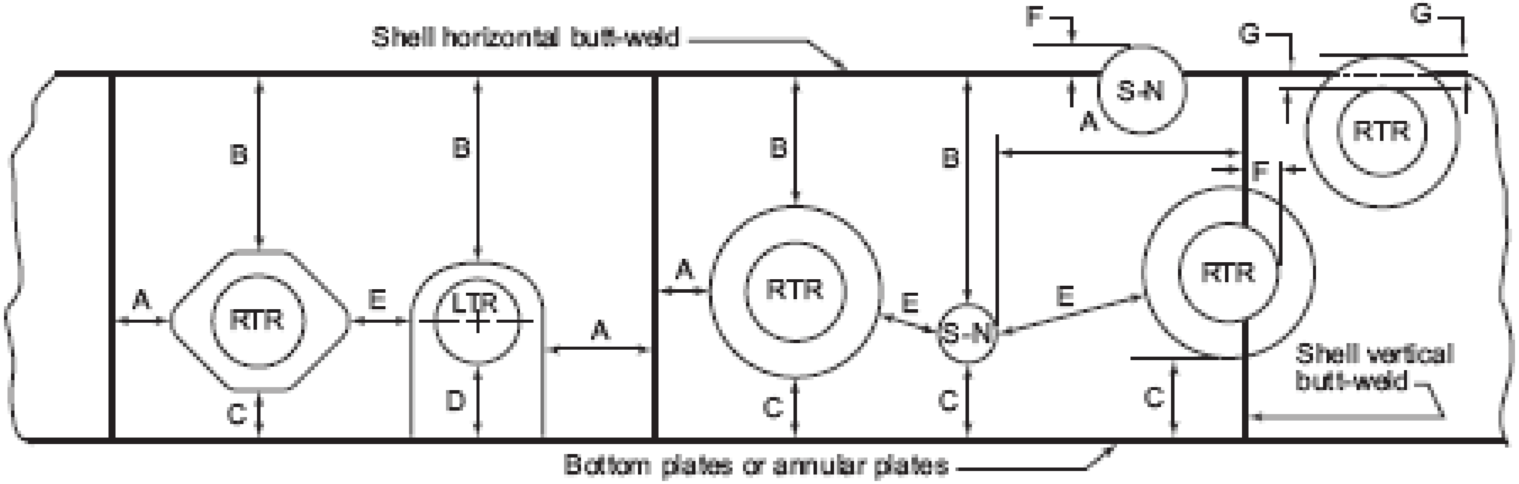
- A shell = Wh x th

- Wh = maximum width of participating roof = Min. (0.3 (R2 th)^0.5 ; 300)

- Where:
- Rc = inside radius of tank shell
- R2 = length of the normal to the roof, measured from the vertical centerline of the tank = Rc / (sin θ)

OPENINGS

Openings



KEY

RTR - Regular-Type Reinforced Opening (nozzle or manhole) with diamond or circular shape reinforcing plate or insert plate that does not extend to the bottom (see Figure 5.7a and Figure 5.8).

LTR - Low-Type Reinforced Opening (nozzle or manhole) using tombstone type reinforcing plate or insert plate that extends to the bottom [see Figure 5.8, Detail (a) and Detail (b)].

S-N - Shell openings with neither a reinforcing plate nor with a thickened insert plate (i.e. integrally reinforced shell openings; or openings not requiring reinforcing).

Variables		Reference	Minimum Dimension Between Weld Toes or Weld Centerline (Notes 1 and 3)						
Shell t	Condition	Paragraph Number	A (2)	B (2)	C (2)	D (3)	E (2)	F (4)	G (4)
$t \le$ 12.5 mm ($t \le$ $^{1}/_{2}$ in.)	As welded or PWHT	5.7.3.2	150 mm (6 in.)	75 mm (3 in.) or $2^{1}/_{2}t$			75 mm (3 in.) or $2^{1}/_{2}t$		
		5.7.3.3			75 mm (3 in.) or $2^{1}/_{2}t$				
		5.7.3.3			75 mm (3 in.) for S-N				
		5.7.3.3				Table 5.6a and Table 5.6b			
		• 5.7.3.4						$8t$ or $^{1}/_{2}$ r	
		• 5.7.3.4							$8t$
$t >$ 12.5 mm ($t >$ $^{1}/_{2}$ in.)	As Welded	5.7.3.1.a	$8W$ or 250 mm (10 in.)	$8W$ or 250 mm (10 in.)					
		5.7.3.1.b					$8W$ or 150 mm (6 in.)		
		5.7.3.3			$8W$ or 250 mm (10 in.)				
		5.7.3.3			75 mm (3 in.) for S-N				
		5.7.3.3				Table 5.6a and Table 5.6b			
		• 5.7.3.4						$8t$ or $^{1}/_{2}$ r	
		• 5.7.3.4							$8t$
$t >$ 12.5 mm ($t >$ $^{1}/_{2}$ in.)	PWHT	5.7.3.2	150 mm (6 in.)	75 mm (3 in.) or $2^{1}/_{2}t$			75 mm (3 in.) or $2^{1}/_{2}t$		
		5.7.3.3			75 mm (3 in.) or $2^{1}/_{2}t$				
		5.7.3.3			75 mm (3 in.) for S-N				
		5.7.3.3				Table 5.6a and Table 5.6b			
		• 5.7.3.4						$8t$ or $^{1}/_{2}$ r	
		• 5.7.3.4							$8t$

NOTE 1 If two requirements are given, the minimum spacing is the greater value, except for dimension "F." See Note 4.

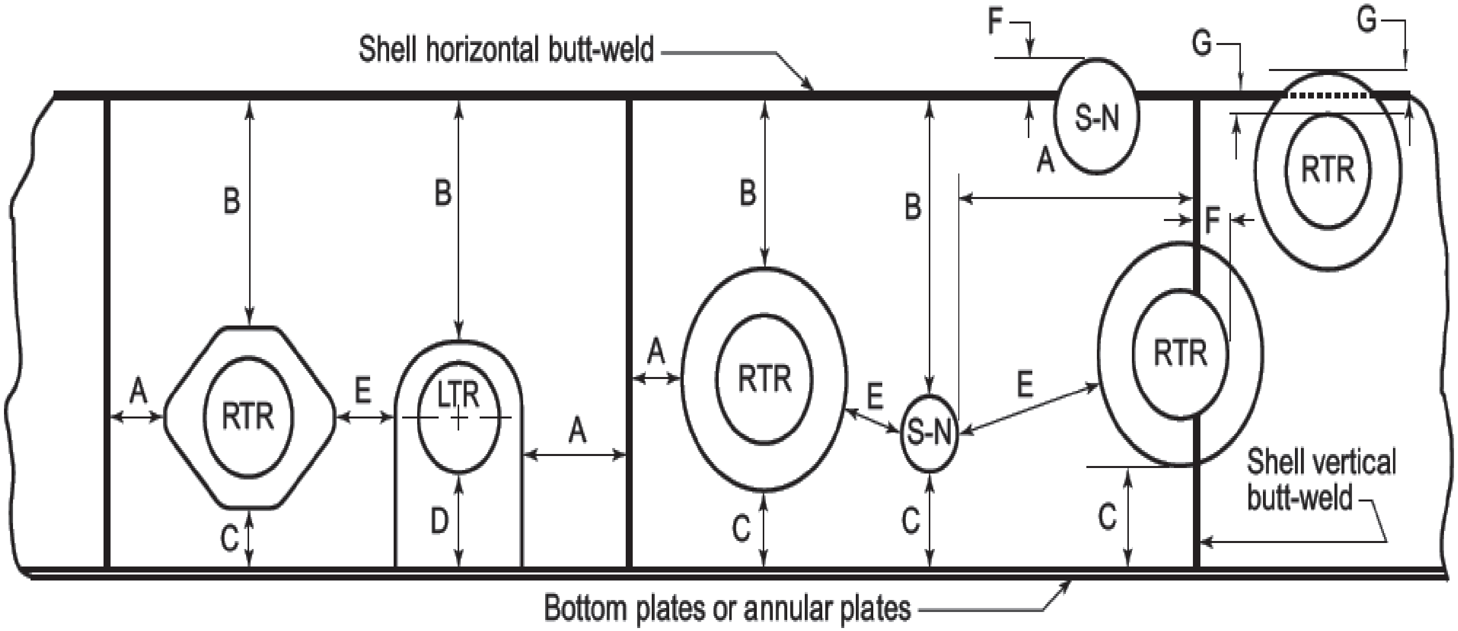
NOTE 2 t = shell thickness. $8W$ = 8 times the largest weld size for reinforcing plate or insert plate periphery weld (fillet or butt-weld) from the toe of the periphery weld to the centerline of the shell butt-weld.

NOTE 3 D = spacing distance established by minimum elevation for low-type reinforced openings from Table 5.6a and Table 5.6b, column 9.

NOTE 4 Purchaser option to allow shell openings to be located in horizontal or vertical shell butt-welds. See Figure 5.9. t = shell thickness, r = radius of opening. Minimum spacing for dimension F is the lesser of $8t$ or $^{1}/_{2}$ r.

Figure 5.6—Minimum Weld Requirements for Openings in Shells According to 5.7.3

Top Angle



KEY

14 **RTR** = Regular-Type Reinforced Opening (nozzle or manhole) with diamond or circular shape reinforcing plate, or insert plate, or thickened
15 insert plate, that does not extend to the bottom (see Figure 5.7A and Figure 5.8).

14 **LTR** = Low-Type Reinforced Opening (nozzle or manhole) using tombstone type reinforcing plate, insert plate, or thickened insert plate that
15 extends to the bottom [see Figure 5.8, Detail (a) and Detail (b)].

14 **S-N** = Shell openings with neither a reinforcing plate nor with a thickened insert plate (i.e. integrally reinforced shell openings; or openings not requiring reinforcing).

Variables		Reference	Minimum Dimension Between Weld Toes or Weld Centerline (Notes 1, 2, 3, and 4)						
Shell t	Condition	Paragraph Number	A	B	C	D (5 only)	E	F (6)	G (6)
$t \le 13$ mm ($t \le 1/2$ in.)	As welded or PWHT	5.7.3.2	150 mm (6 in.)	75 mm (3 in.)			75 mm (3 in.)		
		5.7.3.3			75 mm (3 in.)				
		5.7.3.3							
		5.7.3.3				Table 5.6a and Table 5.6b			
		• 5.7.3.4						Lesser of 8t or $1/2\,r$	
		• 5.7.3.4							8t
$t > 13$ mm ($t > 1/2$ in.)	As Welded	5.7.3.1.a	8W or 250 mm (10 in.)	8W or 250 mm (10 in.)					
		5.7.3.1.b					8W or 150 mm (6 in.)		
		5.7.3.3			8W or 250 mm (10 in.)				
		5.7.3.3			75 mm (3 in.) for S-N				
		5.7.3.3				Table 5.6a and Table 5.6b			
		• 5.7.3.4						Lesser of 8t or $1/2\,r$	
		• 5.7.3.4							8t
$t > 13$ mm ($t > 1/2$ in.)	PWHT	5.7.3.2	150 mm (6 in.)	75 mm (3 in.) or $2\frac{1}{2}t$			75 mm (3 in.) or $2\frac{1}{2}t$		
		5.7.3.3			75 mm (3 in.) or $2\frac{1}{2}t$				
		5.7.3.3			75 mm (3 in.) for S-N				
		5.7.3.3				Table 5.6a and Table 5.6b			
		• 5.7.3.4						Lesser of 8t or $1/2\,r$	
		• 5.7.3.4							8t

NOTE 1 If two requirements are given, the minimum spacing is the greater value, unless otherwise noted.

NOTE 2 Weld spacings are measured to the toe of a fillet-weld, the centerline of an insert or thickened insert plate butt-weld, or the centerline of a shell butt-weld.

NOTE 3 t = shell nominal thickness.

NOTE 4 W = the largest weld size around the periphery of the fitting(s): for fillet welds the leg length along the tank shell, for butt welds the thickness of the insert plate at the weld joint.

NOTE 5 D = spacing distance established by minimum elevation for low-type reinforced openings from Table 5.6a and Table 5.6b, column 9.

NOTE 6 Purchaser option to allow shell openings to be located in horizontal or vertical shell butt-welds. See Figure 5.9.

Tables and Figures

- Shell Manholes:
- Cover Plate and Bolting Flange of shell manhole : Table 5-3
- Dimension of shell manhole neck thickness : Table 5.4
- Dimension of Bolt circle diameter and cover plate diameter of shell manhole: Table 5.5
- Standard figure of shell manhole : Figure 5.7
- Shell Nozzles:
- Dimensions for Shell Nozzles : Table 5.6
- Dimensions for Shell Nozzles: Pipe, Plate, and Welding Schedules (SI) : Table 5.7
- Dimensions for Shell Nozzle Flanges : Table 5.8
- Roof Manholes:
- Dimensions for Roof Manholes: Table 5-13
- Standard figure of roof manhole : Figure 5.16

Tables and Figures

- Drawoff Sump:
- ➢ Standard figure of Sump : Figure 5.21
- ➢ Dimensions for Drawoff Sumps : Table 5.16

- Platforms,Stairways and Walkways:
- ➢ Requirements for Platforms and Walkways : Table 5.17
- ➢ Requirements for Stairways : Table 5.18
- ➢ Rise, Run, and Angle Relationships for Stairways : Table 5.19

- Grounding Lug : Figure 5.23

- Some Acceptable Column Base Details: Figure 5.26

Shell Openings

- 5.7.1.7

✓ Shell openings may be reinforced by the use of an insert plate/reinforcing plate combination or thickened insert plate per Figure 5.7b.

✓ A rectangular insert plate or thickened insert plate shall have rounded corners (except for edges terminating at the tank bottom or at joints between shell courses) with a radius which is greater than or equal to the larger of 150 mm (6 in.) or 6t where t is the thickness of the shell course containing the insert plate or thickened insert plate.

✓ The insert plate or thickened insert plate may contain multiple shell openings.

✓ The thickness and dimensions of insert plate or thickened insert plate shall provide the reinforcing required per 5.7.2.

✓ The periphery of thickened insert plates shall have a 1:4 tapered transition to the thickness of the adjoining shell material when the insert plate thickness exceeds the adjacent shell thickness by more than 3 mm (1/8 in.).

Shell Openings

- 5.7.2.1
- Openings in tank shells larger than required to accommodate a NPS 2 flanged or threaded nozzle shall be reinforced.

- 5.7.2.3

- Reinforcing plates for manholes, nozzles, and other attachments shall be of the same nominal composition (i.e. same ASME P-number and Group Number) as the tank part to which they are attached, unless approved otherwise by the Purchaser

Shell Openings

- 5.7.2.8
- The allowable stresses for the attachment elements are:

➢ a) For outer reinforcing plate-to-shell and inner reinforcing plate-to-nozzle neck fillet welds: Sd × 0.60.

➢ b) For tension across groove welds: Sd × 0.875 × 0.70

➢ c) For shear in the nozzle neck: Sd × 0.80 × 0.875

❖ Sd is the maximum allowable design stress (the lesser value of the base materials joined) permitted by 5.6.2.1 for carbon steel, or by Tables S.2a and S.2b for stainless steel.

❖ The throat of the fillet shall be assumed to be 0.707 times the length of the shorter leg.

Shell Openings

- 5.7.2.9
- When two or more openings are located so that the outer edges (toes) of their normal reinforcing-plate fillet welds are closer than eight times the size of the larger of the fillet welds, with a minimum of 150 mm (6 in.), they shall be treated and reinforced as follows noted in 5.7.2.9 a,b and c.

- 5.7.2.10
- Each reinforcing plate for shell openings shall be provided with a 6 mm (1/4 in.) diameter telltale hole. The hole shall be located on the horizontal centerline and shall be open to the atmosphere.

Shell Openings

- 5.7.3 Spacing of Welds around Connections
- 5.7.3 .1
- a) The toe of the fillet weld around a non-reinforced penetration or around the periphery of a reinforcing plate, and the centerline of a butt-weld around the periphery of a thickened insert plate or insert plate, shall be spaced at least the greater of eight times the weld size or 250 mm (10 in.) from the centerline of any butt-welded shell joints, as illustrated in Figure 5.6, dimensions A or B.

- b) The toe of the fillet weld around a non-reinforced penetration or around the periphery of a reinforcing plate, and the centerline of a butt-weld around the periphery of a thickened insert plate or insert plate, shall be spaced at least the greater of eight times the larger weld size or 150 mm (6 in.) from each other, as illustrated in Figure 5.6, dimension E.

Shell Openings

- 5.7.3 .4
- Nozzles and manholes should not be placed in shell weld seams and reinforcing pads for nozzles and manholes should not overlap plate seams (i.e. Figure 5.9, Details a, c, and e should be avoided). If there is no other feasible option and the Purchaser accepts the design, circular shell openings and reinforcing plates (if used) may be located in a horizontal or vertical butt-welded shell joint provided that the minimum spacing dimensions are met and a radiographic examination of the welded shell joint is conducted. The welded shell joint shall be fully radiographed for a length equal to three times the diameter of the opening, but the weld seam being removed need not be radiographed. Radiographic examination shall be in accordance with 8.1.3 through 8.1.8.

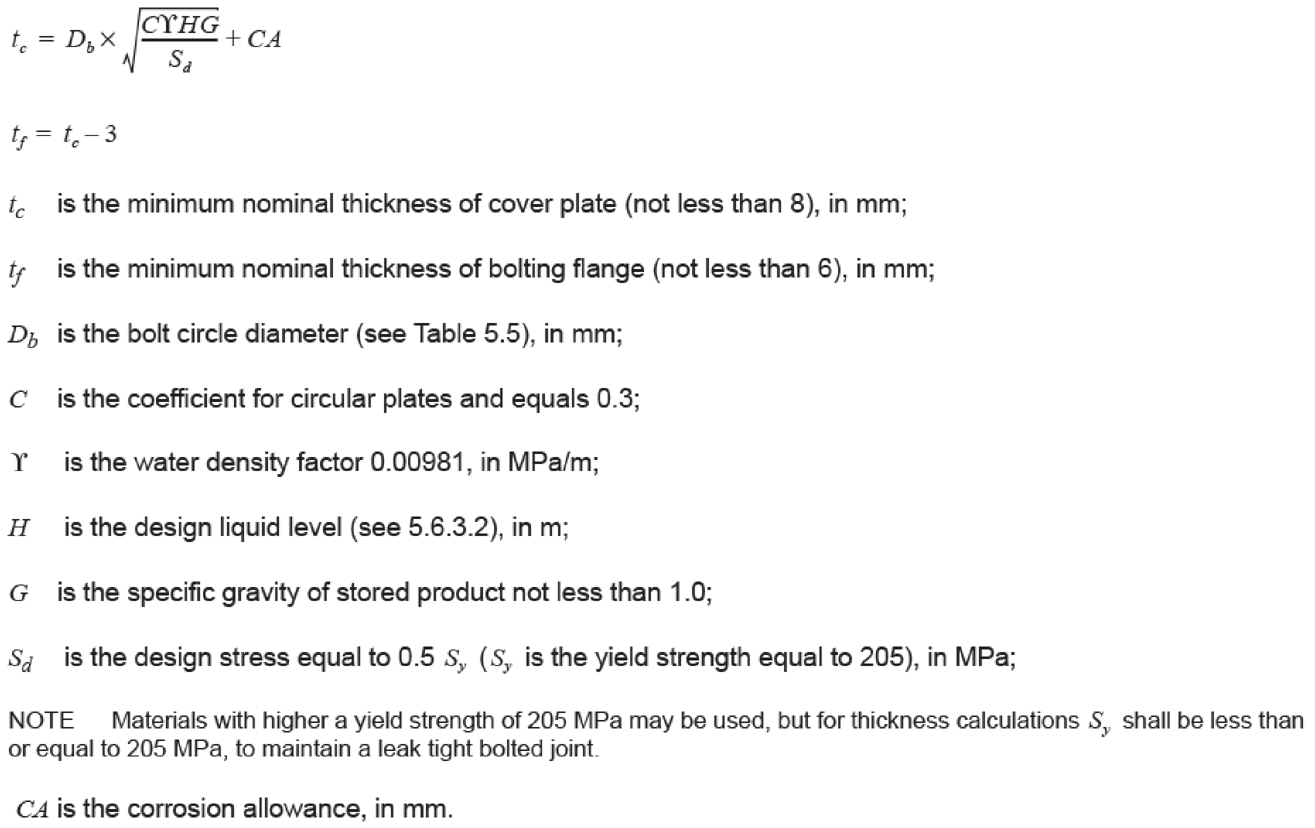
Shell Manholes

- 5.7.5.1
- Each manhole reinforcing plate shall be provided with a 6 mm (1/4 in.) diameter telltale hole (for detection of leakage through the interior welds). The hole shall be located on the horizontal centerline and shall be open to the atmosphere.
- 5.7.5.4
- The gasket materials shall meet service requirements based on the product stored, maximum design temperature, and fire resistance. Gasket dimensions, when used in conjunction with thin-plate flanges described in Figure 5.7a, have proven effective when used with soft gaskets, such as non-asbestos fiber with suitable binder. When using hard gaskets, such as solid metal, corrugated metal, metal-jacketed, and spiral-wound metal, the gasket dimensions, manhole flange, and manhole cover shall be designed per API Standard 620, Section 3.20 and Section 3.21. See 4.9 for additional requirements.

Shell Manholes

5.7.5.6

The required minimum thickness of manhole cover plate and bolting flange shall be the greater of the values computed by the following formulas:



$$t_c = D_b \times \sqrt{\frac{C \Upsilon H G}{S_d}} + CA$$

$$t_f = t_c - 3$$

t_c is the minimum nominal thickness of cover plate (not less than 8), in mm;

t_f is the minimum nominal thickness of bolting flange (not less than 6), in mm;

D_b is the bolt circle diameter (see Table 5.5), in mm;

C is the coefficient for circular plates and equals 0.3;

Υ is the water density factor 0.00981, in MPa/m;

H is the design liquid level (see 5.6.3.2), in m;

G is the specific gravity of stored product not less than 1.0;

S_d is the design stress equal to 0.5 S_y (S_y is the yield strength equal to 205), in MPa;

NOTE Materials with higher a yield strength of 205 MPa may be used, but for thickness calculations S_y shall be less than or equal to 205 MPa, to maintain a leak tight bolted joint.

CA is the corrosion allowance, in mm.

Shell Manholes

EXAMPLE

using a 23 m tall tank with 500 mm manway.

$$t_c = 667 \times \sqrt{\frac{0.3\left(\frac{9.81}{1000}\right)23 \times 1.0}{0.5 \times 205}} + 0 = 17.14\ \text{mm}$$

Wind Load on Tanks (Overturning Stability)

Wind Load on Tanks (Overturning Stability)

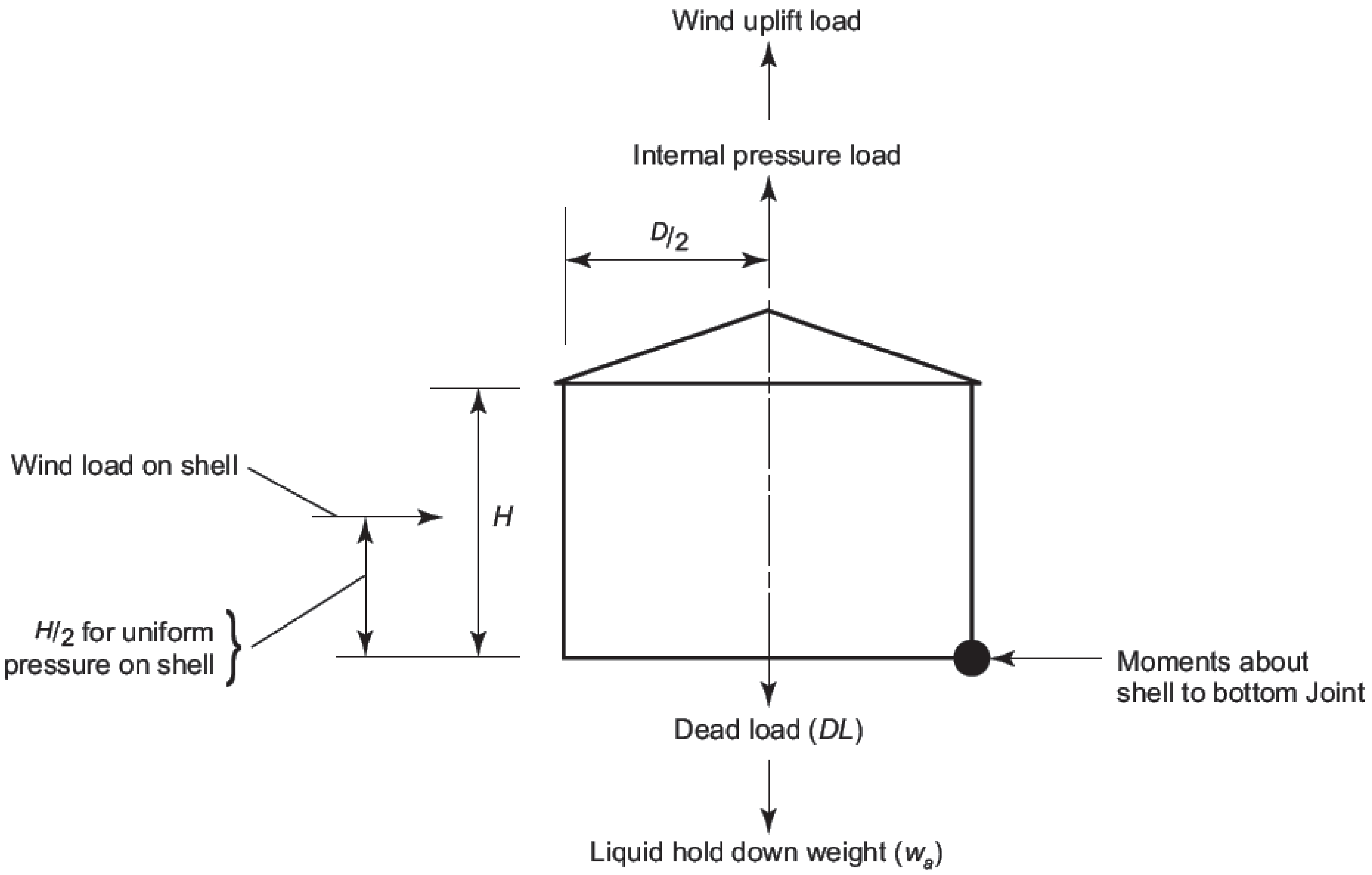
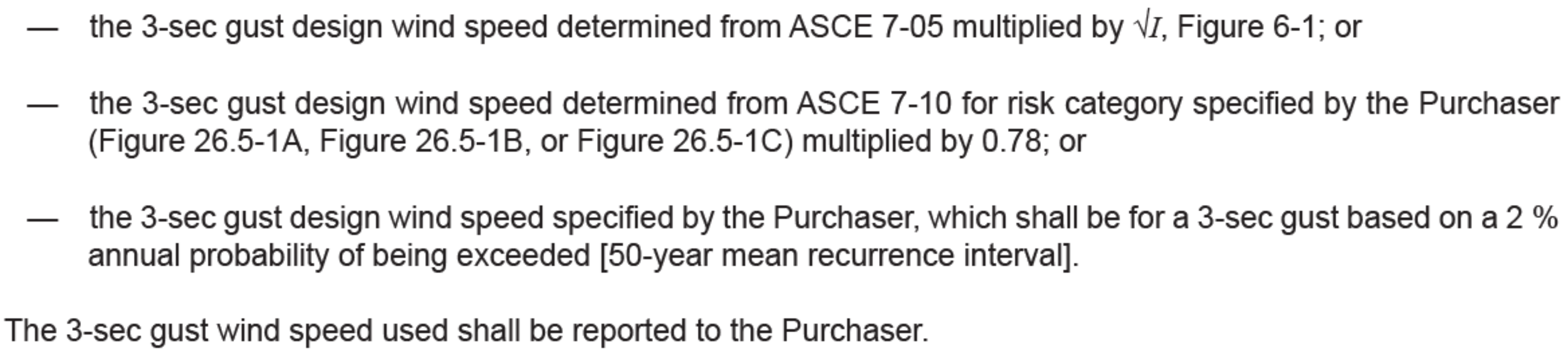


Figure 5.27—Overturning Check for Unanchored Tanks

5.2.1 (K) Wind Load Calculation

- 5.11.1 Overturning stability shall be calculated using the wind pressures given in 5.2.1(k).

- 5.2.1 (K): The design wind speed (V) shall be either:



— the 3-sec gust design wind speed determined from ASCE 7-05 multiplied by $\sqrt{I}$, Figure 6-1; or

— the 3-sec gust design wind speed determined from ASCE 7-10 for risk category specified by the Purchaser (Figure 26.5-1A, Figure 26.5-1B, or Figure 26.5-1C) multiplied by 0.78; or

— the 3-sec gust design wind speed specified by the Purchaser, which shall be for a 3-sec gust based on a 2 % annual probability of being exceeded [50-year mean recurrence interval].

The 3-sec gust wind speed used shall be reported to the Purchaser.

5.2.1 (K) Wind Load Calculation

- 1) Design wind pressure (PWS and PWR) using design wind speed (V): The design wind pressure on shell (PWS) shall be 0.86 kPa (V/190)2, on vertical projected areas of cylindrical surfaces. The design wind uplift pressure on roof (PWR) shall be 1.44 kPa (V/190)2 (see item 2) on horizontal projected areas of conical or doubly curved surfaces. These design wind pressures are in accordance with ASCE 7-05 for wind exposure Category C. As alternatives, pressures may be determined in accordance with:
- a) ASCE 7-05 (exposure category and importance factor provided by Purchaser); or
- b) ASCE 7-10 (exposure category and risk category provided by Purchaser) with either velocity multiplied by 0.78 or the ASCE 7-10 pressure multiplied by 0.6; or
- c) a national standard for the specific conditions for the tank being designed.

5.2.1 (K) Wind Load Calculation

- 2) The design uplift pressure on the roof (wind plus internal pressure) need not exceed 1.6 times the design pressure P determined in F.4.1.

- 3) Windward and leeward horizontal wind loads on the roof are conservatively equal and opposite and therefore they are not included in the above pressures.

- 4) Fastest mile wind speed times 1.2 is approximately equal to 3-sec gust wind speed (V).

Note:

ASCE 7-10 wind velocities now have LRFD load factors and risk category (importance factors) built in, whereas API 650 uses the working stress. The 0.78 factor applied to the ASCE 7-10 wind speed provides a conversion to working stress levels.

Wind Load on Tanks (Overturning Stability)

- Unanchored tanks shall meet the requirements of 5.11.2.1 or 5.11.2.2.
- When the requirements of 5.11.2 cannot be satisfied, anchor the tank per the requirements of 5.12.

- 5.11.2.1 Unanchored tanks, except supported cone roof tanks meeting the requirements of 5.10.4, shall satisfy all of the following uplift criteria:

1) $0.6M_w + M_{Pi} < M_{DL} /1.5 + M_{DLR}$

2) $M_w + F_p(M_{Pi}) < (M_{DL} + M_F)/2 + M_{DLR}$

3) $M_{ws} + F_p\ (M_{Pi}) < M_{DL} /1.5 + M_{DLR}$

Wind Load on Tanks (Overturning Stability)

- 5.11.2.2 Unanchored tanks with supported cone roofs meeting the requirements of 5.10.4 shall satisfy the following criteria:

$$M_{ws} + F_p\,(M_{Pi}) < M_{DL}/1.5 + M_{DLR}$$

Wind Load on Tanks (Overturning Stability)

where

F_P	is the pressure combination factor, see 5.2.2;
M_{Pi}	is the moment about the shell-to-bottom joint from design internal pressure;
M_w	is the overturning moment about the shell-to-bottom joint from horizontal plus vertical wind pressure;
M_{DL}	is the moment about the shell-to-bottom joint from the nominal weight of the shell and roof structure supported by the shell that is not attached to the roof plate;
M_F	is the moment about the shell-to-bottom joint from liquid weight;
M_{DLR}	is the moment about the shell-to-bottom joint from the nominal weight of the roof plate plus any attached structural;
M_{WS}	is the overturning moment about the shell-to-bottom joint from horizontal wind pressure.

Wind Load on Tanks (Overturning Stability)

- 5.11.2.3 wL is the resisting weight of the tank contents per unit length of shell circumference based on a specific gravity (G) of 0.7 or the actual product specific gravity, whichever is less, and a height of one-half the design liquid height H. wL shall be the lesser of 70.4 HD for SI Units (0.45 HD for USC units) or the following:

$$w_L = 70 t_b \sqrt{(F_{by} G H)} \text{ (N/m)}$$

Wind Load on Tanks (Overturning Stability)

where

F_{by} is the minimum specified yield stress of the bottom plate under the shell, in MPa (lbf/in.^{2});

G is the actual specific gravity of the stored liquid or 0.7, whichever is less;

H is the design liquid height, in meters (ft);

D is the tank diameter, in meters (ft);

t_b is the required corroded thickness of the bottom plate under the shell, in mm (inches), that is used to resist wind overturning. The bottom plate shall have the following restrictions:

1) The corroded thickness, t_b, used to calculate w_L shall not exceed the first shell course corroded thickness less any shell corrosion allowance.

2) When the bottom plate under the shell is thicker due to wind overturning than the remainder of the tank bottom, the minimum projection of the supplied thicker annular ring inside the tank wall, L, shall be the greater of 450 mm (18 in.) or L_b, however, need not be more than $0.035D$.

Wind Load on Tanks (Overturning Stability)

- 1) The corroded thickness, tb, used to calculate wL shall not exceed the first shell course corroded thickness less any shell corrosion allowance.
- 2) When the bottom plate under the shell is thicker due to wind overturning than the remainder of the tank bottom, the minimum projection of the supplied thicker annular ring inside the tank wall, L, shall be the greater of 450 mm (18 in.) or Lb, however, need not be more than 0.035D.

$$L_b = 0.024\ t_b\sqrt{(F_{by}/(GH))} \le 0.035\ D\ \text{(in meters)}$$

Design data:

I.D = 6 m

H = 7.2 m

HHLL = 6.7 m

Specific gravity = 1.483

C.A. = 3 mm

Course Width = 1800 mm

DESIGN WIND VELOCITY: 125 Km/hr

Snow load: 0 KPa

DP = Full of Liquid

DT = 85°c

MAT = A 283 C
- Sd = 137 MPa
- St = 154 MPa
- Fy = 205 MPa

Bottom Plate: 6mm + C.A. = $\boxed{9\text{mm}}$

SHELL DESIGN (1-one feet Method):

$$t_N = \text{Max}\begin{cases} t_{d_N} = \dfrac{4.9D(H-0.3)G}{S_d} + CA \\[2ex] t_{t_N} = \dfrac{4.9D(H-0.3)}{S_t} \end{cases}$$

$$t_1 = \text{Max}\begin{cases} t_{d_1} = \dfrac{4.9D(H-0.3)G}{S_d} + CA = \dfrac{4.9(6)(7.2-0.3)\times 1.483}{137} + 3 = 5.19\ \text{mm} \\[2ex] t_{t_1} = \dfrac{4.9D(H-0.3)}{S_t} = \dfrac{4.9(6)(7.2-0.3)}{154} = 1.37\ \text{mm} \end{cases}$$

So, Selected thk = Max (t_{d_1}, t_{t_1}) = 5.19

Min API Std. thk. = 6mm For First shell Course
5mm For other Courses
} As PER 5.6.1.1

So, SELECTED THICKNESS IS: 6mm

$$t_2 = \left\{ \begin{array}{l} t_{d2} = \dfrac{4.9 \times 6 \times (7.2 - 1.8 - 0.3) \times 1.483}{137} + 3 = 4.623 \text{ mm} \\ \\ t_{t2} = \dfrac{4.9 \times 6 \times (7.2 - 1.8 - 0.3)}{154} = 0.97 \text{ mm} \end{array} \right\} t_2 = 5 \text{ mm}$$

FOR OTHER COURSES SAME PROCEDURE TO BE DON

SO COURSE # 3 & 4 = 5mm

WEIGHT CALCULATION FOR SHELL

W = π (D + thk) × thk × DENSITY × 1.04 × SHELL HEIGHT

1st = $\pi (6 + 0.006) \times 0.006 \times 7850 \times 1.04 \times 1.8 \cong 1{,}663.65$ kg

FOR OTHER COURSES = $\pi (6 + 0.005) \times 0.005 \times 7850 \times 1.8 \times 1.04 \cong 1{,}386$ Kg

So, TOTAL SHELL COURSE WEIGHT IS = $\boxed{5{,}821.65 \text{ Kg}}$

ANNULAR PLATE WIDTH & THK.

Product Stress = $\dfrac{(td - CA)}{t_{corr}} (Sd) = \dfrac{(5.19 - 3)}{3} (137) = \boxed{100.01 \text{ MPa}}$

Hydrostatic Test Stress : $(t_t / t_{nominal}) (S_t) = \dfrac{1.37}{6} (154) = 35.1$ MPa

So, Stress in First shell Course is 100 MPa

and thk of $t \leq 19 \longrightarrow$ So, Annular plate thk. is $= 6 + 3 = 9$ mm

Annular plate width :

$= \text{Max.}(600 ; L) + t_{shell} + \text{overlap length}(5t) + 50\text{ mm}$

$$\boxed{= \text{Max.}(600 ; 563.07) + 6 + 50 + 50 = 706\text{ mm}}$$

$$L = 2 t_b \sqrt{\frac{F_y}{2 \gamma G H}} = 2 \times 9\text{mm} \times \sqrt{\frac{205}{2\left(\frac{9.81}{1000}\right) \times 1.483 \times 7.2}}$$

$L = 563.07$ mm

Roof Calculation

$$t = \frac{D}{4.8 \sin\theta} \sqrt{\frac{B}{2.2}} + CA$$ or 5mm whichever is greater

$$B = \text{Max.} \begin{cases} D_L + (L_r) + F_{pe} P_e \Rightarrow \\ D_L + P_e + 0.4(L_r) \end{cases}$$

④

D_L = weight of roof and any other attachments / roof area

$$\frac{\text{Roof Weight}}{\text{Roof Area}} : \frac{\pi \frac{D^2}{4} \times \text{Density} \times \text{thk} \times 1.04}{\pi \frac{D^2}{4}} \cong \text{Density} \times \text{thk}$$

$$\text{So, } D_L = 7850 \times 0.012\ \text{m} = 94.2 \times 0.01 \cong 0.95\ \text{KPa} \simeq 1\text{KP}$$

(سایر اوزان نیز می‌بایست در برآورد بالا در نظر گرفته می‌شد که به علت کمی !! ساده‌سازی از آن صرف‌نظر کرده)

$$\begin{cases} D_L + L_r + 0.4 P_e = 1\text{KPa} + 1\text{KPa} + 0 = 2\ \text{KPa} \\ \\ D_L + 0.4 L_r + P_e = 1\text{KPa} + 0.4\ 1\text{KPa} = 1.4\ \text{KPa} \end{cases}$$

$$\text{So, } B(T) = 2\text{KPa} \quad , \theta =$$

$$t = \frac{6}{4.8 \sin(12)} \sqrt{\frac{2}{2.2}} = 5.74\ \text{mm}$$

با فرض اولیه 12mm ضخامت برای سقف تناقض ندارد و بنابراین 12mm اولیه مورد قبول است.

حالا اگر ورق 6 جواب بده چرا 12mm ؟!! چون بسیاری از بارها را در مراحل ساده‌سازی کردیم

Participating area

$$A_{req} = \frac{PD^2}{8 F_a \tan\theta} = \frac{2(6)^2}{8(123)(0.21)} = \boxed{0.34\ mm^2}$$

the same diffication with $B(T) \cong 2\ kPa$

$F_a = 0.6F_y = 0.6 \times 205 = 123\ MPa$

$\theta = 12°$

Detail $\underline{b}$

A_{ava} as per F.2 is : $A_{ava} = A_{shell} + A_{roof} + A_{angle}$

$A_{shell} = W_c \times t_s = 73.48mm \times 5mm = 367.42\ mm^2$

$A_{roof} = W_h \times t_h = 124.8\ mm \times 12mm = 1498mm^2$

$A_{angle} = ?$ $\quad A_{shell} + A_{roof} > A_{req} \rightarrow$ So, angle

$W_c = 0.6\sqrt{R_c t} = 0.6\sqrt{(5)(3000)} = 73.48\ mm$

all dimensions are in mm

$W_h = Min\left[0.3\sqrt{R_2 t_h} ; 300\right] = Min\left[0.3\sqrt{14429.2 \times 12} ; 300\right] = 124.8\ mm$

$R_2 = \frac{R_c}{\sin\theta} = \frac{3000}{\sin(12)} = 14429.2\ mm$

→ Could be Min. size as per API 650

$\boxed{50 \times 50 \times 5}$

WIND GIRDER CALCULATION

$$H_1 = 9.47\,t\sqrt{\left(\frac{t}{D}\right)^3}\left(\frac{190}{V}\right)^2 = 9.47\left(\underset{2}{\cancel{5}}\right)\sqrt{\left(\frac{2}{6}\right)^3}\left(\frac{190}{150}\right)^2 =$$

should be considered as corroded 2mm

$$V = 125\ \frac{km}{hr} \times \underset{\text{as per ASCE 7}}{1.2} = 150\ \frac{km}{hr}.$$

$$\Rightarrow H_1 = 5.848\ m$$

Transformed shell:

$$W_{tr} = W\sqrt{\left(\frac{t_{un}}{t_{ac}}\right)^5}$$

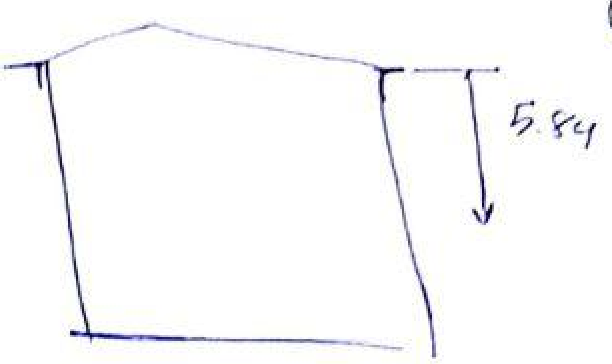
$$W_{tr_1} = 1800\sqrt{\left(\frac{(5-3)}{(6-3)}\right)^5} = 1800 \times \sqrt{\left(\frac{2}{3}\right)^5} = 1084.32\ mm$$

$$W_{tr_{2,3,4}} = 1800\sqrt{\left(\frac{2}{2}\right)^5} = 1800$$

$$\sum W_{tr} = 6.484\ m$$

SINCE $\sum W_{tr}$ IS MORE THAN H_1, SO ONE INTERMEDIATE WIND GIRDER IS REQUIRED

POSITION OF WIND GIRDER



THE EXACT LOCATION CAN BE CALCULATED BASED ON REVERSE FORMULA AND A RING WILL BE PLACED AT FIRST COURSE, BUT FOR MORE STABILITY (AS PER API) WE PUT THIS RING AT ABOUT MIDDLE HEIGHT OF TANK

THEREFORE A RING WILL BE PLACED AT 4000 mm.

SIZE OF RING

$$Z = \frac{D^2 h_1}{17}\left(\frac{V}{190}\right)^2 = \frac{6^2(3.2)}{17}\left(\frac{150}{190}\right)^2 = 4.22 \text{ cm}^3$$

FROM TABLE 5.20a, Detail C WE CHOOSE $\boxed{65 \times 65 \times 6}$

WIND STABILITY

WIND PRESSURE:

ON SHELL = $0.86\,\text{KPa}\left(\frac{150}{190}\right)^2 = 0.536\,\text{KPa}$

ON ROOF = $1.44\,\text{KPa}\left(\frac{150}{190}\right)^2 = 0.897\,\text{KPa}$

WIND FORCE (SHEAR)

ON SHELL = $P_{W_S} \times \pi D H = 72.74\ \text{N}$

ON ROOF = $P_{W_R} \times \pi D^2/4 = 25.36\ \text{N}$

WIND MOMENT

$$F_S \times H/2 + F_R \times D/2 = 261.86\ \text{N}\cdot\text{m} + 76.08\ \text{N}\cdot\text{m}$$

$$\boxed{= 337.94\ \text{N}\cdot\text{m}}$$

5.11.2.1:

1) $0.6\,M_W + M_{Pi} < M_{DL}/1.5 + M_{DLR}$

2) $M_W + F_P(M_{Pi}) < (M_{DL} + M_F)/2 + M_{DLR}$

3) $M_{WS} + F_P(M_{Pi}) < M_{DL}/1.5 + M_{DLR}$

$M_w = 337,94$ N.m

$F_p = 0.4$

$M_{P_i} = 0$

$M_{DL} =$ Shell Weight $\times \frac{D}{2} = 5821.65 \times 3 = 17464.95$ Kg × m

$M_{DL} = 171156.51$ N.m

$M_{DLR} = 3046.97 \times 3 = 9140.93$ Kg.m $= 89581.15$

$M_F = W_L \times \frac{D}{2} = 1145.13$ N.m

$$W_L = \text{Min} \begin{cases} 70 t_b \sqrt{(F_{by} G H)} \times \pi D = \boxed{381.7 \text{ N}} \\ 70.4\, HD = 3041.28 \text{ N} \end{cases}$$

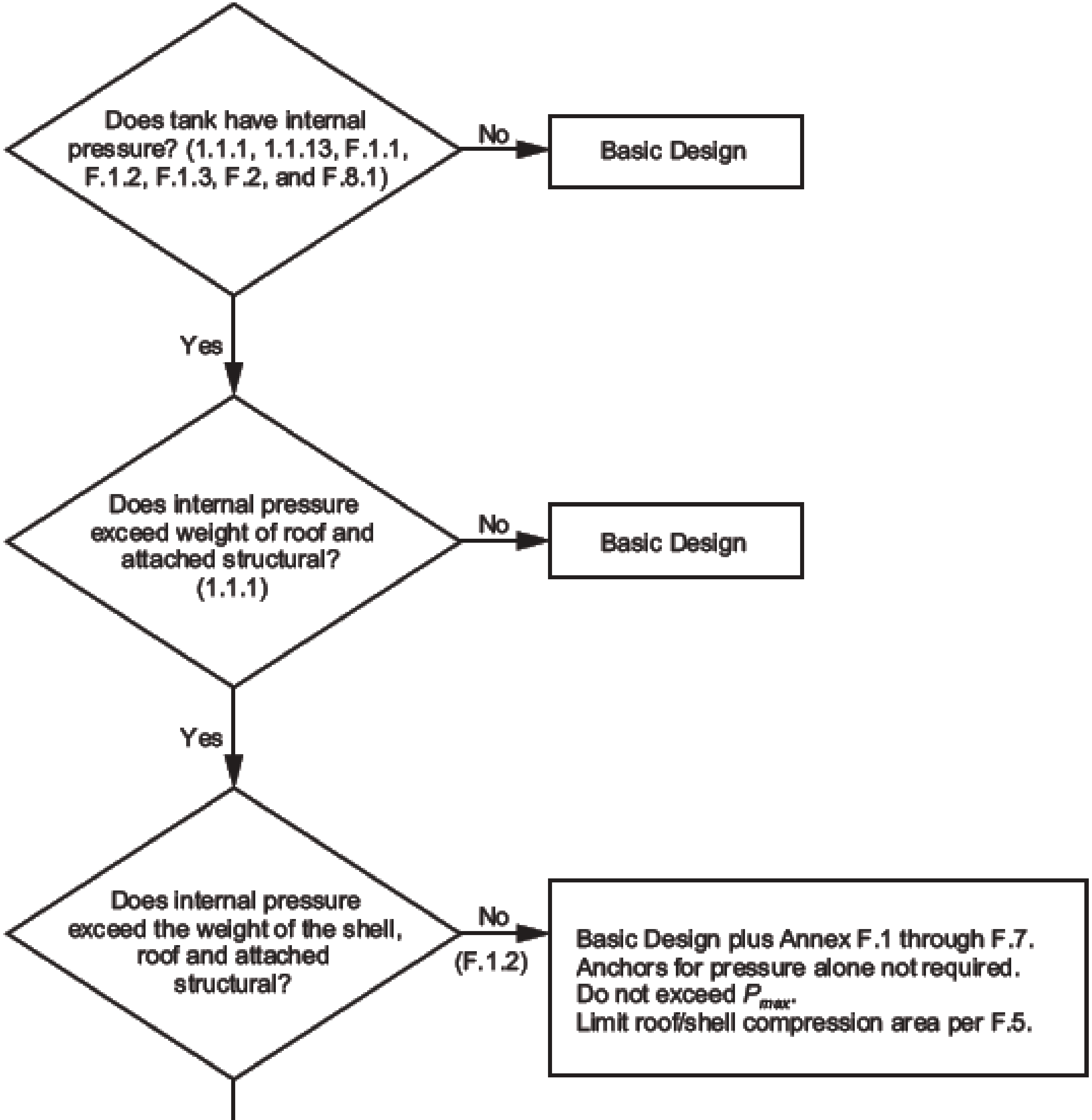
$$\Rightarrow \begin{cases} 1)\ 0.6 \times 337.94 + 0 < \frac{171156.51}{1.5} + 89581.15 & \text{Pass!} \\ 2)\ 337.94 + 0 < \left(\frac{171156.51 + 1145.13}{2}\right) + 89581.15 & \text{Pass!} \\ 3)\ \ldots\ldots & \text{Pass!} \end{cases}$$

$\boxed{\text{So, TANK IS STABLE}}$

App. F (INTERNAL PRESSURE)

App. F

- F.1.1 The maximum internal pressure for closed-top API Standard 650 tanks may be increased to the maximum internal pressure permitted (18 kPa [2.5 lbf/in.2]) gauge when the additional requirements of this Annex are met. This Annex applies to the storage of nonrefrigerated liquids (see also API 620, Annex Q and Annex R). For maximum design temperatures above 93 °C (200 °F), see Annex M.



Does tank have internal pressure? (1.1.1, 1.1.13, F.1.1, F.1.2, F.1.3, F.2, and F.8.1)

No

Basic Design

Yes

Does internal pressure exceed weight of roof and attached structural? (1.1.1)

No

Basic Design

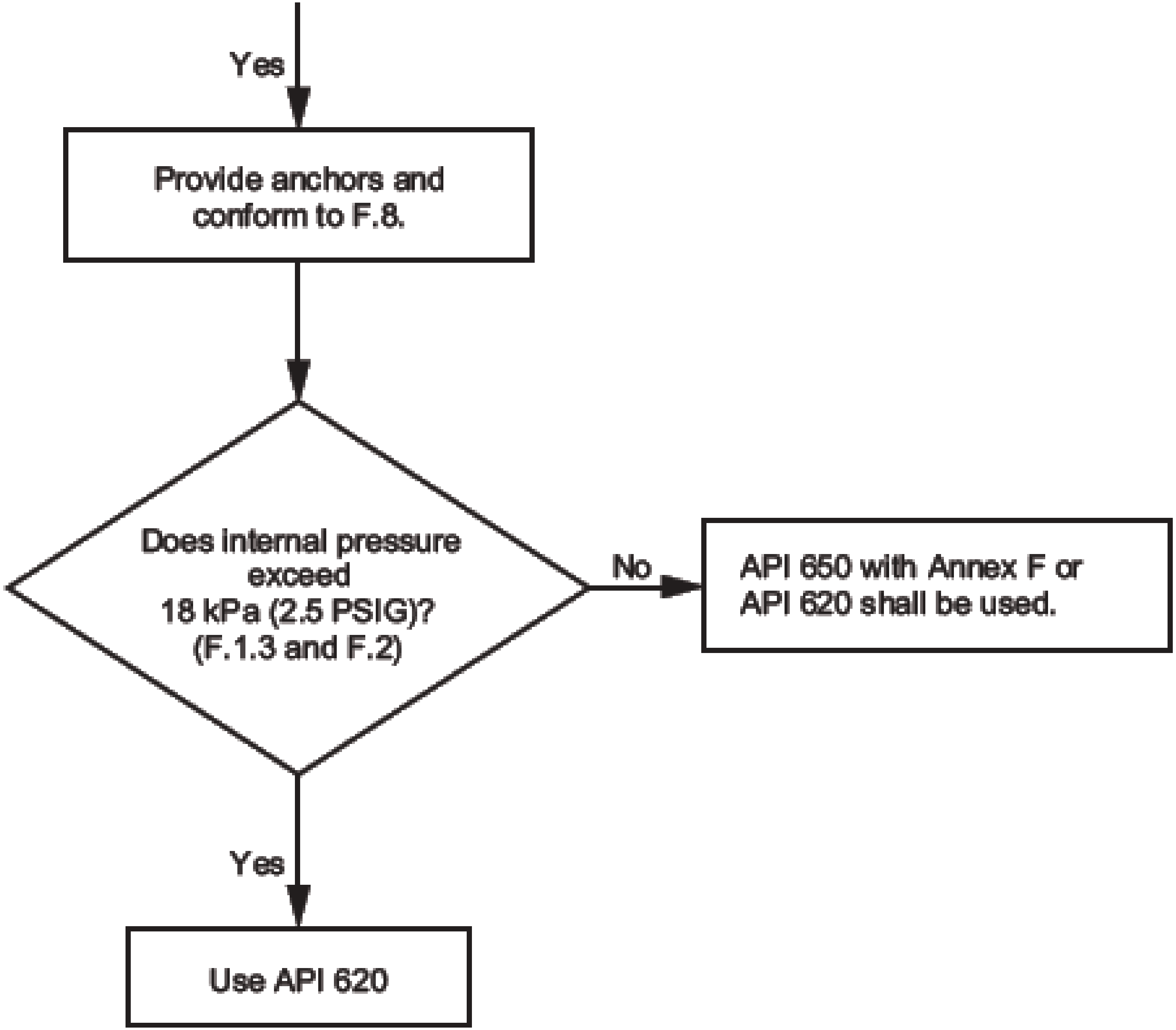
Yes

Does internal pressure exceed the weight of the shell, roof and attached structural?

No

(F.1.2)

Basic Design plus Annex F.1 through F.7.
Anchors for pressure alone not required.
Do not exceed P_{max}.
Limit roof/shell compression area per F.5.



Yes

Provide anchors and conform to F.8.

Does internal pressure exceed 18 kPa (2.5 PSIG)? (F.1.3 and F.2)

No

API 650 with Annex F or API 620 shall be used.

Yes

Use API 620

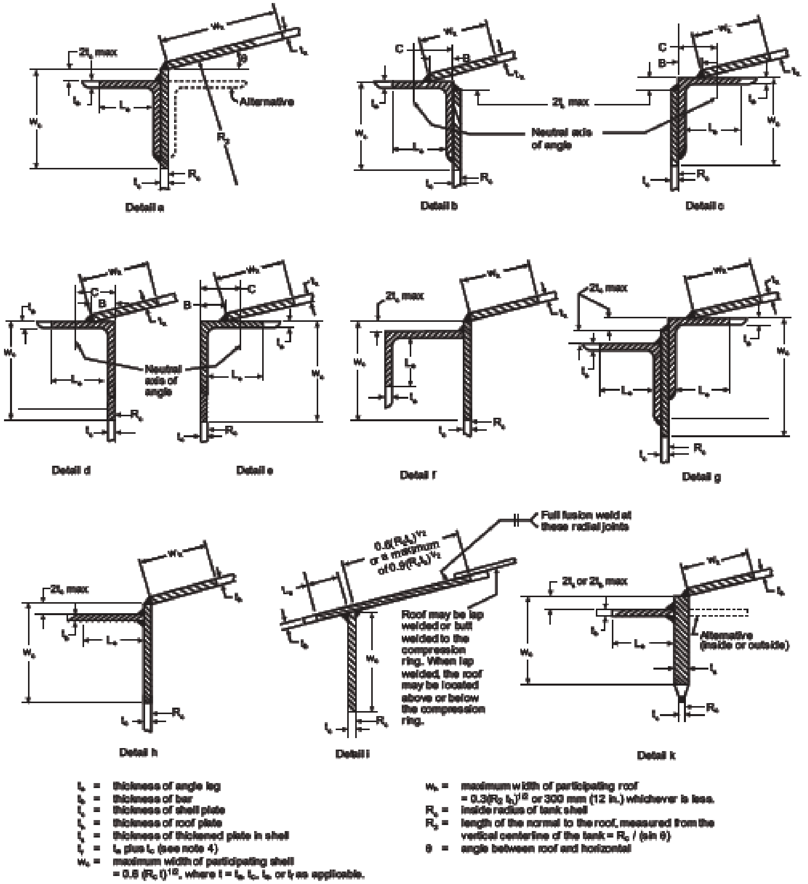
App. F

- F.2 Design Considerations
- F.2.1 In calculating shell thickness for Annex F tanks and when selecting shell manhole thicknesses in Table 5.3a and Table 5 3b and flush-type cleanout fitting thicknesses in Table 5.10a and Table 5.10b, H shall be increased by the quantity P/(9.8G) for SI units, or P/(12G) for USC units—where H is the design liquid height, in m (ft), P is the design pressure kPa (in. of water), and G is the design specific gravity. Design pressures less than 1 kPa (4 in. of water) do not need to be included.

App. F

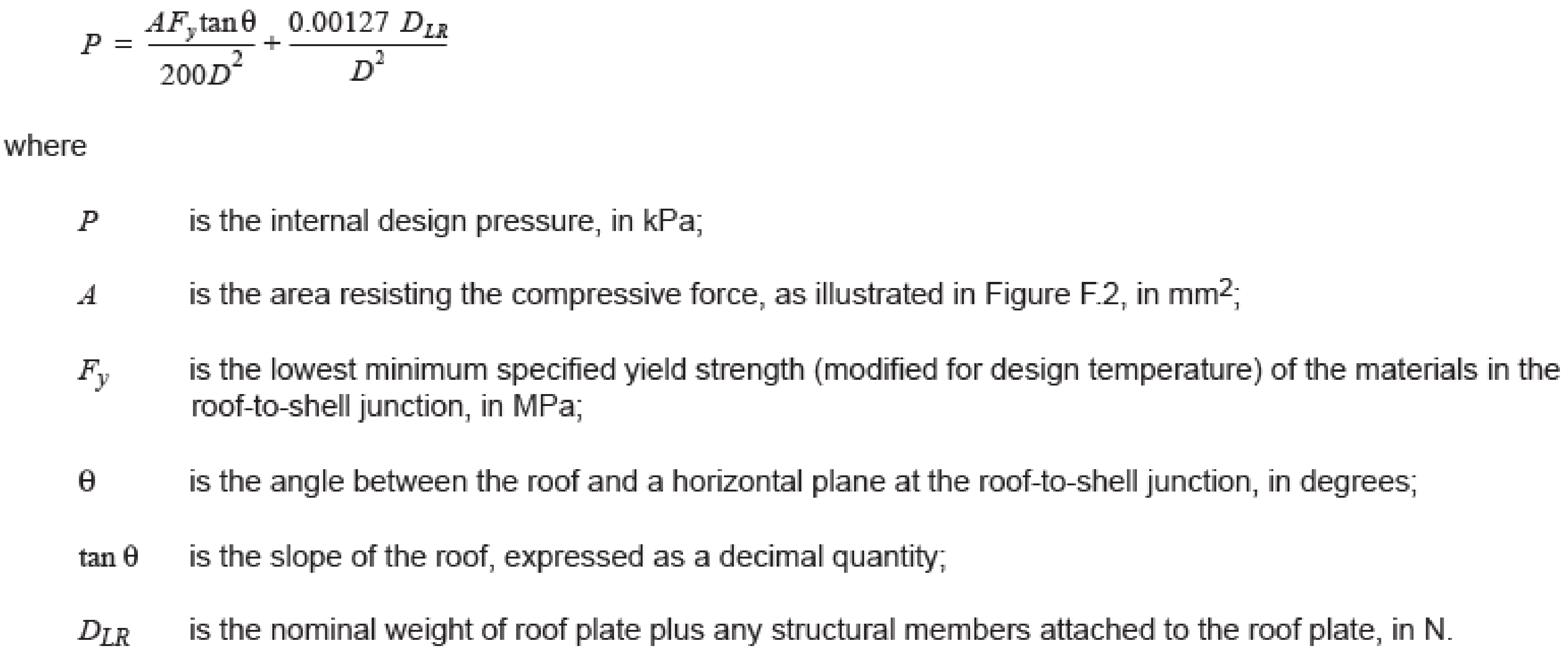
- F.3 Roof Details

The details of the roof-to-shell junction shall be in accordance with Figure F.2, in which the participating area resisting the compressive force is shaded with diagonal lines.



App. F

- **F.4 Maximum Design Pressure and Test Procedure**
- F.4.1 The maximum design pressure, P, for a tank that has been constructed or that has had its design details established may be calculated from the following equation (subject to the limitations of Pmax in F.4.2):



$$P = \frac{AF_y \tan\theta}{200D^2} + \frac{0.00127\ D_{LR}}{D^2}$$

where

P	is the internal design pressure, in kPa;
A	is the area resisting the compressive force, as illustrated in Figure F.2, in mm^2;
F_y	is the lowest minimum specified yield strength (modified for design temperature) of the materials in the roof-to-shell junction, in MPa;
θ	is the angle between the roof and a horizontal plane at the roof-to-shell junction, in degrees;
$\tan\theta$	is the slope of the roof, expressed as a decimal quantity;
D_{LR}	is the nominal weight of roof plate plus any structural members attached to the roof plate, in N.

App. F

- F.4.2 For unanchored tanks, the maximum design pressure, limited by uplift at the base of the shell, shall not exceed the value calculated from the following equations as applicable unless further limited by F.4.3:
- For unanchored fixed roof tanks except supported cone roof tanks, the maximum design pressure (Pmax) shall be the minimum of (3) cases:

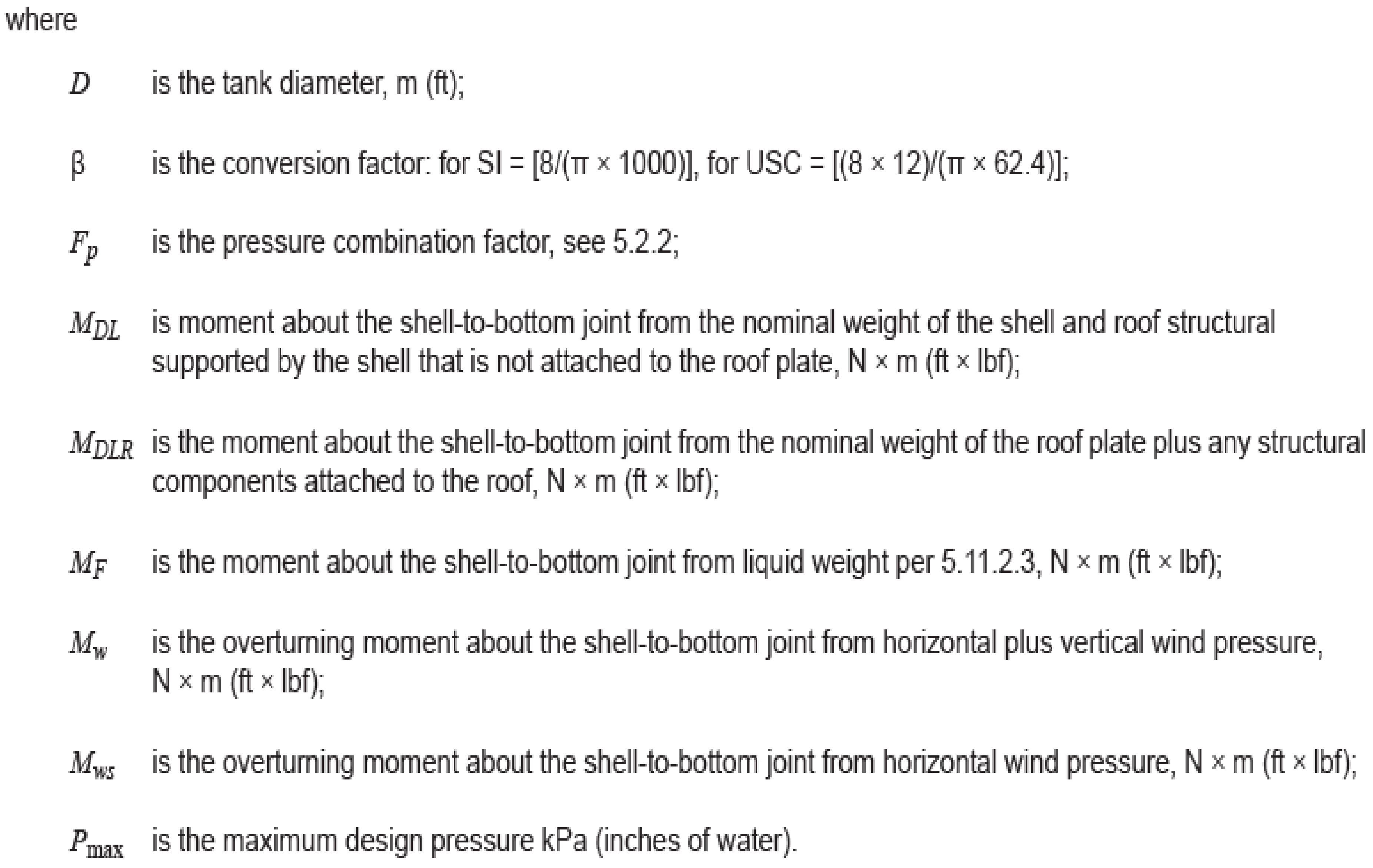
$$(1) \qquad \frac{\beta}{D^3}\left(\frac{M_{DL}}{1.5} + M_{DLR} - 0.6\, M_w\right)$$

$$(2) \qquad \frac{\beta}{F_p \cdot D^3}\left(\frac{M_{DL} + M_F}{2} + M_{DLR} - M_w\right)$$

$$(3) \qquad \frac{\beta}{F_p \cdot D^3}\left(\frac{M_{DL}}{1.5} + M_{DLR} - M_{ws}\right)$$

For unanchored supported cone roof tanks:

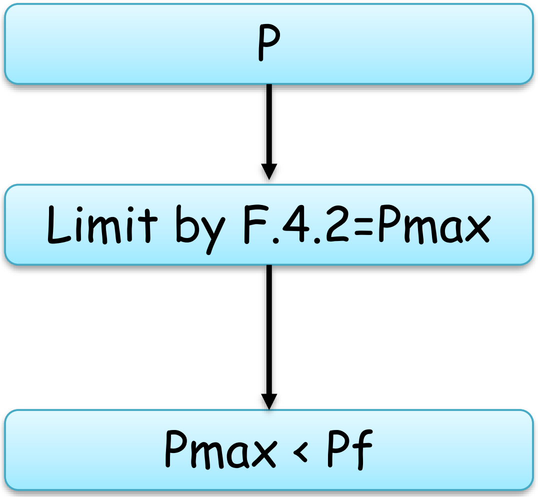
$$P_{\max} = \frac{\beta}{F_p \cdot D^3}\left(\frac{M_{DL}}{1.5} + M_{DLR} - M_{ws}\right)$$



where

D	is the tank diameter, m (ft);
β	is the conversion factor: for SI = [8/(π × 1000)], for USC = [(8 × 12)/(π × 62.4)];
F_p	is the pressure combination factor, see 5.2.2;
M_{DL}	is moment about the shell-to-bottom joint from the nominal weight of the shell and roof structural supported by the shell that is not attached to the roof plate, N × m (ft × lbf);
M_{DLR}	is the moment about the shell-to-bottom joint from the nominal weight of the roof plate plus any structural components attached to the roof, N × m (ft × lbf);
M_F	is the moment about the shell-to-bottom joint from liquid weight per 5.11.2.3, N × m (ft × lbf);
M_w	is the overturning moment about the shell-to-bottom joint from horizontal plus vertical wind pressure, N × m (ft × lbf);
M_{ws}	is the overturning moment about the shell-to-bottom joint from horizontal wind pressure, N × m (ft × lbf);
P_{max}	is the maximum design pressure kPa (inches of water).

App. F



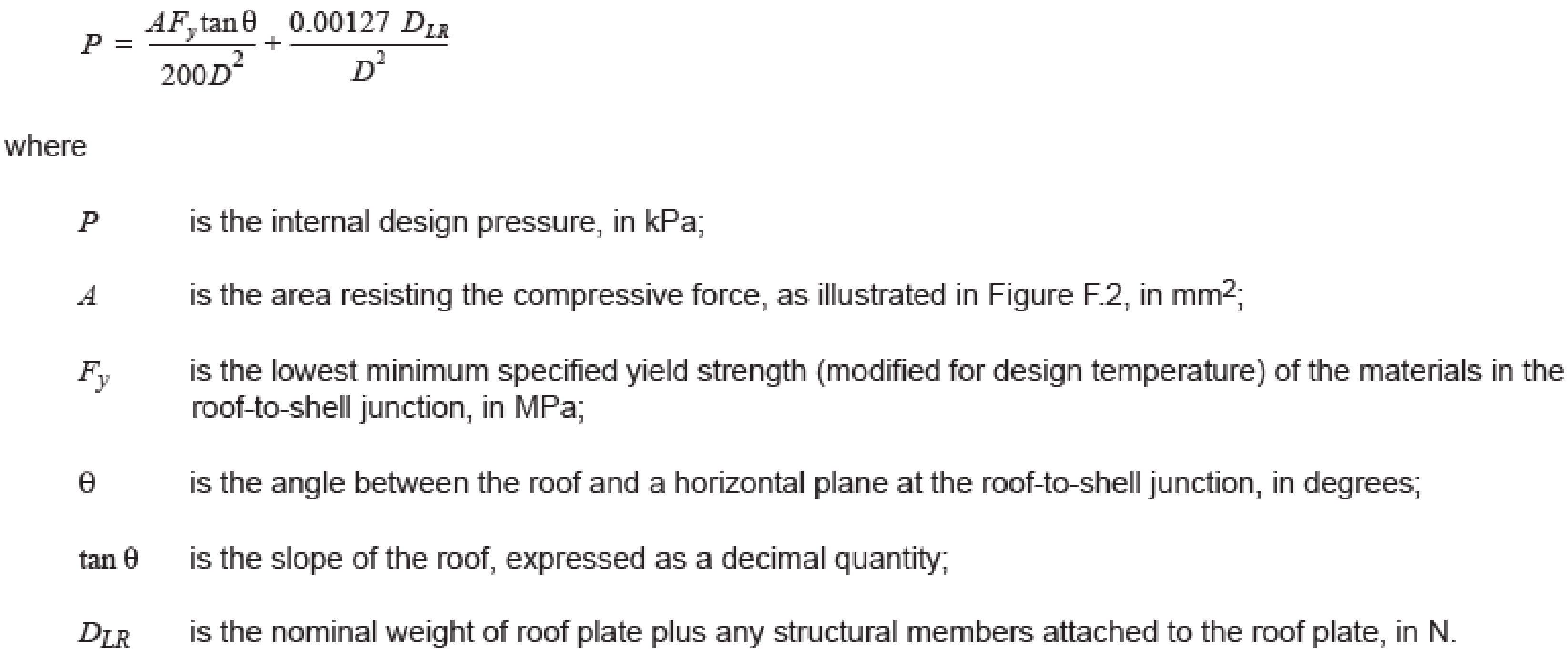
P

Limit by F.4.2=Pmax

Pmax < Pf

App. F

- **F.4 Maximum Design Pressure and Test Procedure**
- F.4.1 The maximum design pressure, P, for a tank that has been constructed or that has had its design details established may be calculated from the following equation (subject to the limitations of Pmax in F.4.2):



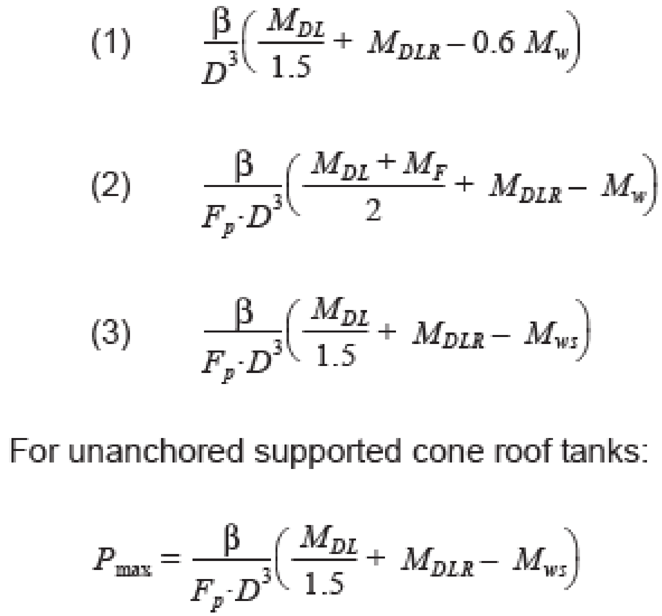
$$P = \frac{AF_y\tan\theta}{200D^2} + \frac{0.00127\ D_{LR}}{D^2}$$

where

P	is the internal design pressure, in kPa;
A	is the area resisting the compressive force, as illustrated in Figure F.2, in mm^2;
F_y	is the lowest minimum specified yield strength (modified for design temperature) of the materials in the roof-to-shell junction, in MPa;
θ	is the angle between the roof and a horizontal plane at the roof-to-shell junction, in degrees;
$\tan\theta$	is the slope of the roof, expressed as a decimal quantity;
D_{LR}	is the nominal weight of roof plate plus any structural members attached to the roof plate, in N.

App. F

- F.4.2 For unanchored tanks, the maximum design pressure, limited by uplift at the base of the shell, shall not exceed the value calculated from the following equations as applicable unless further limited by F.4.3:
- For unanchored fixed roof tanks except supported cone roof tanks, the maximum design pressure (Pmax) shall be the minimum of (3) cases:



(1) $$\frac{\beta}{D^3}\left(\frac{M_{DL}}{1.5} + M_{DLR} - 0.6\,M_w\right)$$

(2) $$\frac{\beta}{F_p \cdot D^3}\left(\frac{M_{DL} + M_F}{2} + M_{DLR} - M_w\right)$$

(3) $$\frac{\beta}{F_p \cdot D^3}\left(\frac{M_{DL}}{1.5} + M_{DLR} - M_{ws}\right)$$

For unanchored supported cone roof tanks:

$$P_{\max} = \frac{\beta}{F_p \cdot D^3}\left(\frac{M_{DL}}{1.5} + M_{DLR} - M_{ws}\right)$$

where

D is the tank diameter, m (ft);

β is the conversion factor: for SI = $[8/(\pi \times 1000)]$, for USC = $[(8 \times 12)/(\pi \times 62.4)]$;

F_p is the pressure combination factor, see 5.2.2;

M_{DL} is moment about the shell-to-bottom joint from the nominal weight of the shell and roof structural supported by the shell that is not attached to the roof plate, N × m (ft × lbf);

M_{DLR} is the moment about the shell-to-bottom joint from the nominal weight of the roof plate plus any structural components attached to the roof, N × m (ft × lbf);

M_F is the moment about the shell-to-bottom joint from liquid weight per 5.11.2.3, N × m (ft × lbf);

M_w is the overturning moment about the shell-to-bottom joint from horizontal plus vertical wind pressure, N × m (ft × lbf);

M_{ws} is the overturning moment about the shell-to-bottom joint from horizontal wind pressure, N × m (ft × lbf);

P_{max} is the maximum design pressure kPa (inches of water).

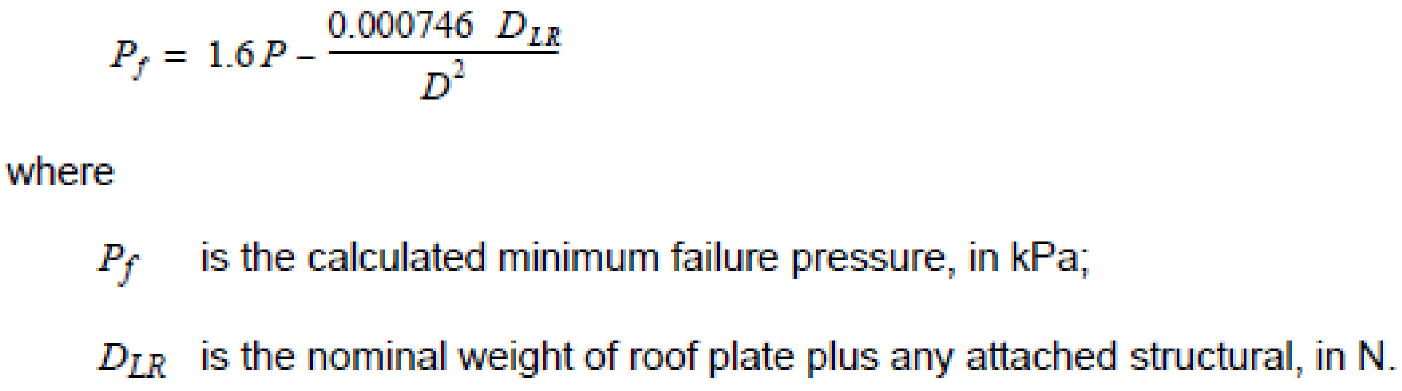
App. F

- F.4.3 As top angle size and roof slope decrease and tank diameter increases, the design pressure permitted by F.4.1 and F.4.2 approaches the failure pressure of F.7 for the roof-to-shell junction. In order to provide a safe margin between the maximum operating pressure and the calculated failure pressure, a suggested further limitation on the maximum design pressure for tanks with a weak roof-to-shell attachment (frangible joint) is: $P_{\max} \leq 0.8P_f$

App. F

- ## F.7 Calculated Failure Pressure

Failure of the roof-to-shell junction can be expected to occur when the stress in the compression ring area reaches the yield point. On this basis, an approximate formula for the pressure at which failure of the top compression ring is expected (using conservative effective areas) to occur can be expressed in terms of the design pressure permitted by F.4.1, as follows:



$$P_f = 1.6P - \frac{0.000746\ D_{LR}}{D^2}$$

where

P_f is the calculated minimum failure pressure, in kPa;

D_{LR} is the nominal weight of roof plate plus any attached structural, in N.

App. F

- F.4.4 When the entire tank is completed, it shall be filled with water to the top angle or the design liquid level, and the design internal air pressure shall be applied to the enclosed space above the water level and held for 15 minutes. The air pressure shall then be reduced to one-half the design pressure, and all welded joints above the liquid level shall be checked for leaks by means of a soap film, linseed oil, or another suitable material. Tank vents shall be tested during or after this test.

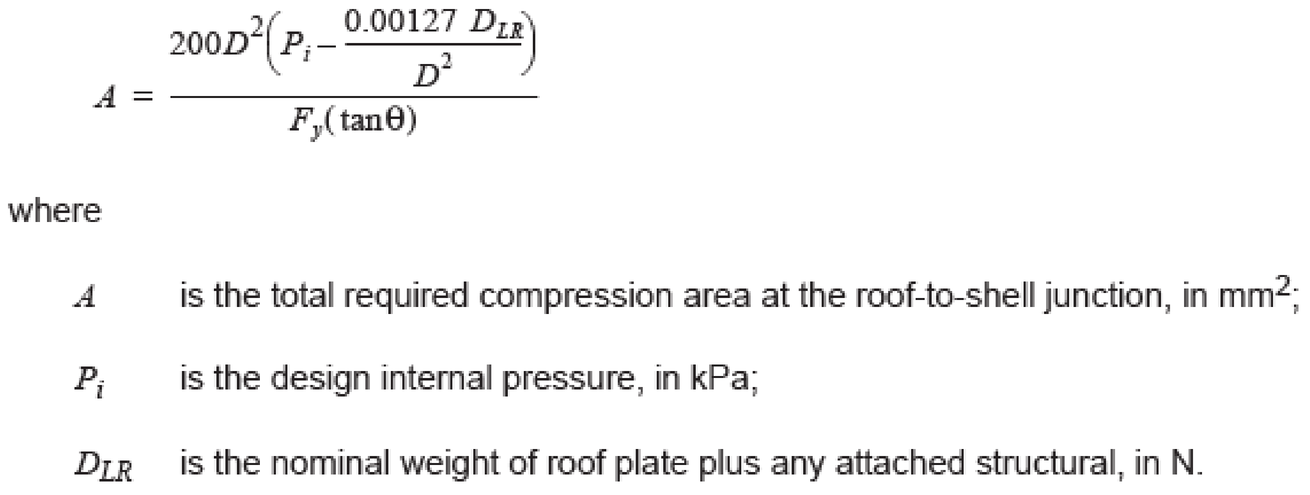
App. F

- F.8.3 After the tank is filled with water, the shell and the anchorage shall be visually inspected for tightness. Air pressure of 1.25 times the design pressure shall be applied to the tank filled with water to the design liquid height. The air pressure shall be reduced to the design pressure, and the tank shall be checked for tightness. In addition, all seams above the water level shall be tested using a soap film or another material suitable for the detection of leaks.
- After the test water has been emptied from the tank (and the tank is at atmospheric pressure), the anchorage shall be checked for tightness. The design air pressure shall then be applied to the tank for a final check of the anchorage.

App. F

- F.5 Required Compression Area at the Roof-to-Shell Junction

- F.5.1 Where the maximum design pressure has already been established (not higher than that permitted by F.4.2 or F.4.3, whenever applicable), the total required compression area at the roof-to-shell junction shall be calculated from the following equation:



$$A = \frac{200D^2\left(P_i - \frac{0.00127\ D_{LR}}{D^2}\right)}{F_y(\tan\theta)}$$

where

A is the total required compression area at the roof-to-shell junction, in mm^2;

P_i is the design internal pressure, in kPa;

D_{LR} is the nominal weight of roof plate plus any attached structural, in N.

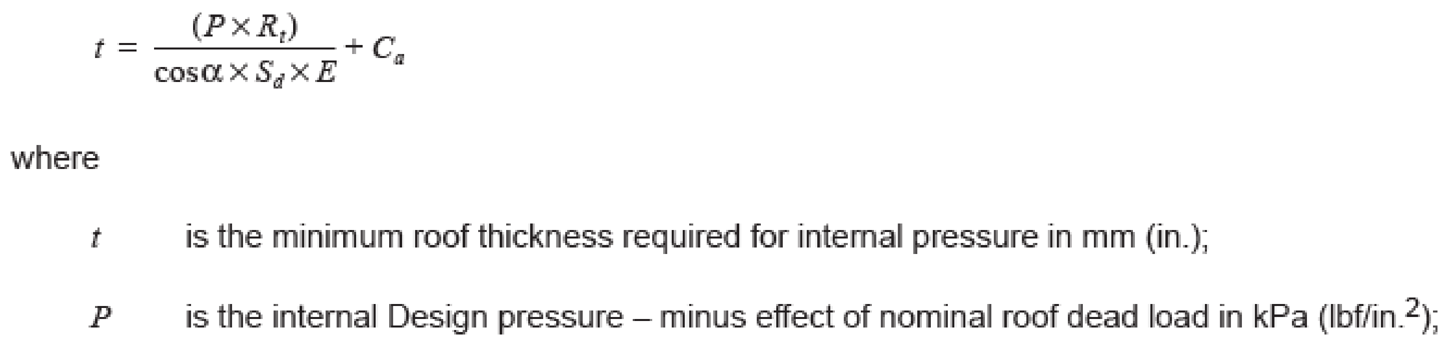
App. F

F.6 Design of Roof Plates

- F.6.1 Minimum thickness of supported and self-supporting cone roofs under internal pressure shall be calculated as follows:

➢ NOTE 1 Thickness (t) of lap welded plates when controlled by internal pressure design shall not exceed 13 mm (1/2 in.) excluding corrosion allowance.

➢ NOTE 2 Calculated thickness (t) of roof plates shall not be less than that required under 5.10.4 for supported cone or less than that required under 5.10.5 for self-supporting cone roofs.



$$t = \frac{(P \times R_t)}{\cos\alpha \times S_d \times E} + C_a$$

where

t is the minimum roof thickness required for internal pressure in mm (in.);

P is the internal Design pressure – minus effect of nominal roof dead load in kPa (lbf/in.^{2});

R_t is the nominal tank radius in m (in.);

a is the half apex angle of cone roof (degrees);

$\cos\alpha$ is the cosine of half apex angle expressed as a decimal quantity;

S_d is the allowable stress for the design condition per this Standard in MPa, (lbf/in.^{2});

E is the joint efficiency:

E = 0.35 for full fillet lap welded plate from top side only,

E = 0.65 for full fillet lap welded plate from both sides,

E = 0.70 for full-penetration, complete-fusion butt welded plates with or without backing strip,

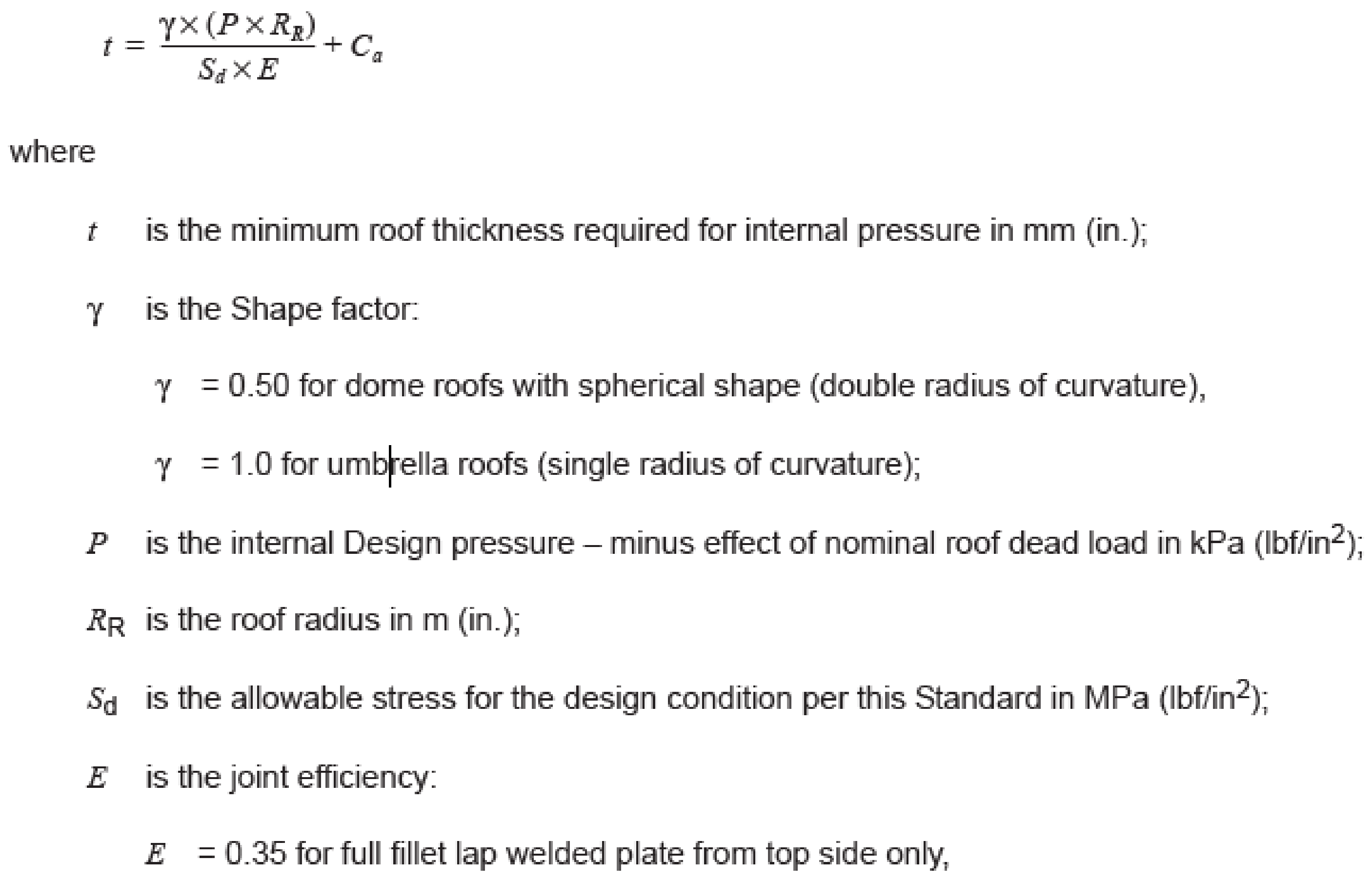
E = 0.85 for full-penetration, complete-fusion butt welded plates with spot radiography in accordance with 8.1.2.2,

E = 1.0 for full-penetration, complete-fusion butt welded plates with 100% full radiography;

C_a is the corrosion allowance in mm (in.) as specified by the Purchaser (see 5.3.2).

App. F

- F.6.2 Minimum thickness of self-supporting dome and umbrella roofs under internal pressure shall be calculated as follows:



$$t = \frac{\gamma \times (P \times R_R)}{S_d \times E} + C_a$$

where

t is the minimum roof thickness required for internal pressure in mm (in.);

γ is the Shape factor:

γ = 0.50 for dome roofs with spherical shape (double radius of curvature),

γ = 1.0 for umbrella roofs (single radius of curvature);

P is the internal Design pressure – minus effect of nominal roof dead load in kPa (lbf/in^2);

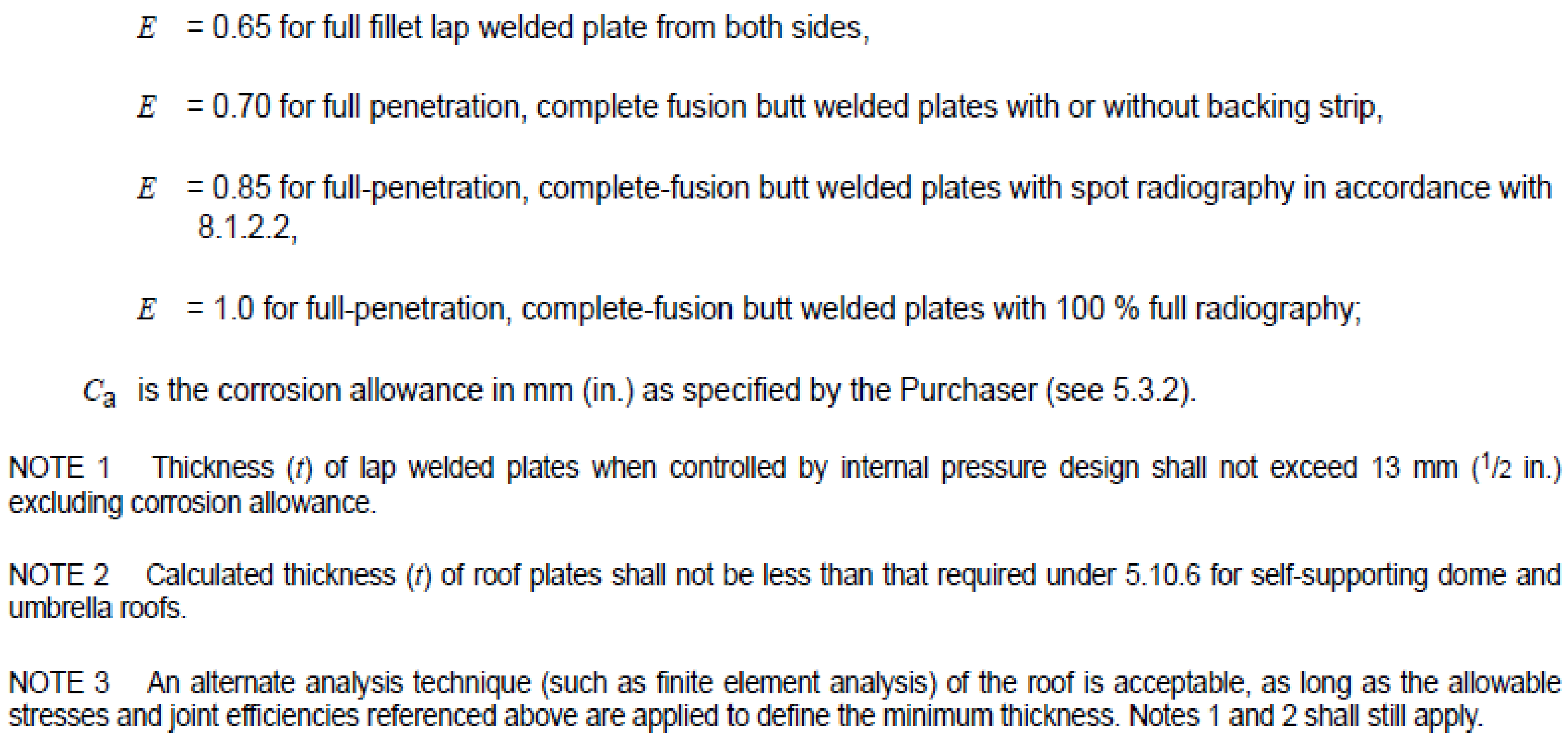
R_R is the roof radius in m (in.);

S_d is the allowable stress for the design condition per this Standard in MPa (lbf/in^2);

E is the joint efficiency:

E = 0.35 for full fillet lap welded plate from top side only,

App. F



E = 0.65 for full fillet lap welded plate from both sides,

E = 0.70 for full penetration, complete fusion butt welded plates with or without backing strip,

E = 0.85 for full-penetration, complete-fusion butt welded plates with spot radiography in accordance with 8.1.2.2,

E = 1.0 for full-penetration, complete-fusion butt welded plates with 100 % full radiography;

C_a is the corrosion allowance in mm (in.) as specified by the Purchaser (see 5.3.2).

NOTE 1 Thickness (t) of lap welded plates when controlled by internal pressure design shall not exceed 13 mm ($^1/2$ in.) excluding corrosion allowance.

NOTE 2 Calculated thickness (t) of roof plates shall not be less than that required under 5.10.6 for self-supporting dome and umbrella roofs.

NOTE 3 An alternate analysis technique (such as finite element analysis) of the roof is acceptable, as long as the allowable stresses and joint efficiencies referenced above are applied to define the minimum thickness. Notes 1 and 2 shall still apply.

- F.6.3 The rules in F.6.1 and F.6.2 cannot cover all details of tank roof design and construction. With the approval of the Purchaser, the roof need not comply with F.6. The manufacturer shall provide a roof designed and constructed to be as safe as otherwise provided for in this standard.

App. F

- F.8 Anchored Tanks with Design Pressures up to 18 kPa (2.5 psi) Gauge
- F.8.1 The design of the anchorage and its attachment to the tank shall be a matter of agreement between the Manufacturer and the Purchaser and shall meet the requirements of 5.12.
- F.8.2 The counterbalancing weight, in addition to the requirements in 5.12, shall be designed so that the resistance to uplift at the bottom of the shell will be the greatest of the following.
- a) The uplift produced by 1.5 times the design pressure of the corroded empty tank plus the uplift from the design wind velocity on the tank.
- b) The uplift produced by 1.25 times the test pressure applied to the empty tank (with the nominal thicknesses).
- c) The uplift produced by 1.5 times the calculated failure pressure (Pf in F.6) applied to the tank filled with the design liquid. The effective weight of the liquid shall be limited to the inside projection of the ringwall (Annex B type) from the tank shell. Friction between the soil and the ringwall may be included as resistance. When a footing is included in the ringwall design, the effective weight of the soil may be included.

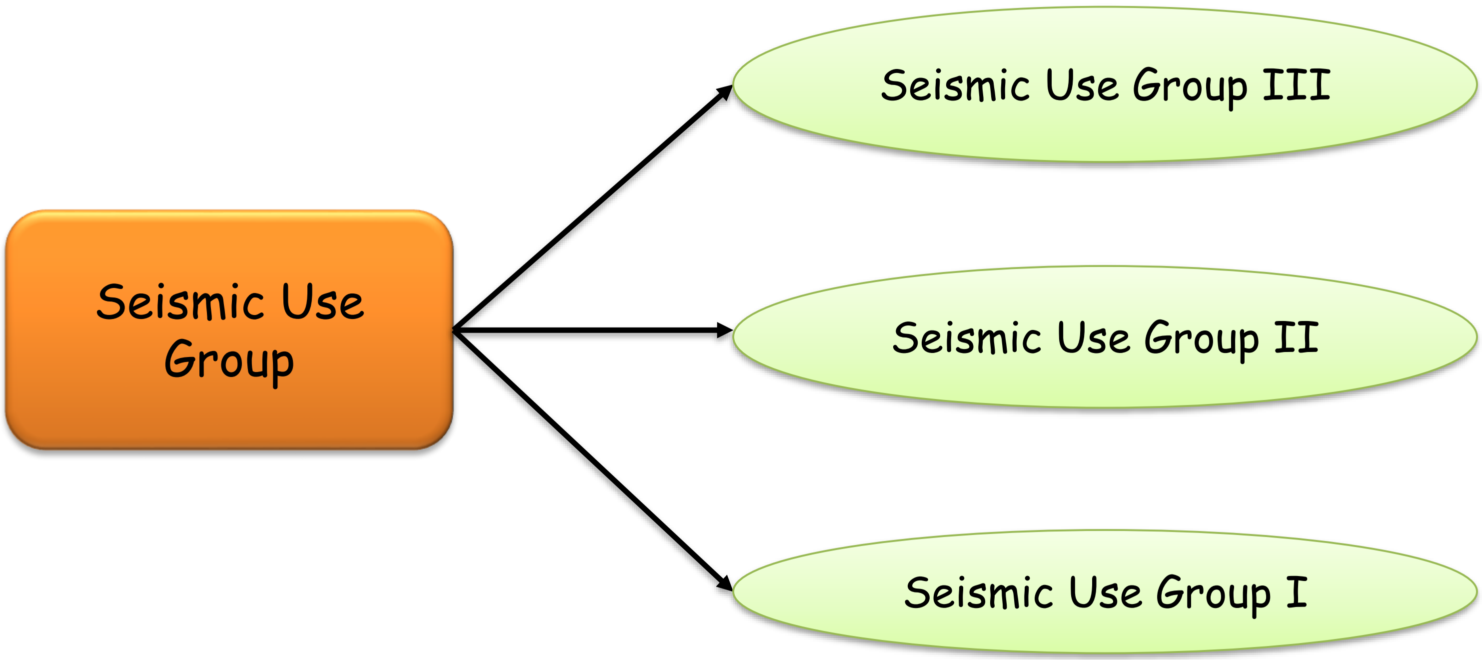
App. E (SEISMIC DESIGN OF STORAGE TANKS)

App. E (SEISMIC DESIGN OF STORAGE TANKS)

AIMS

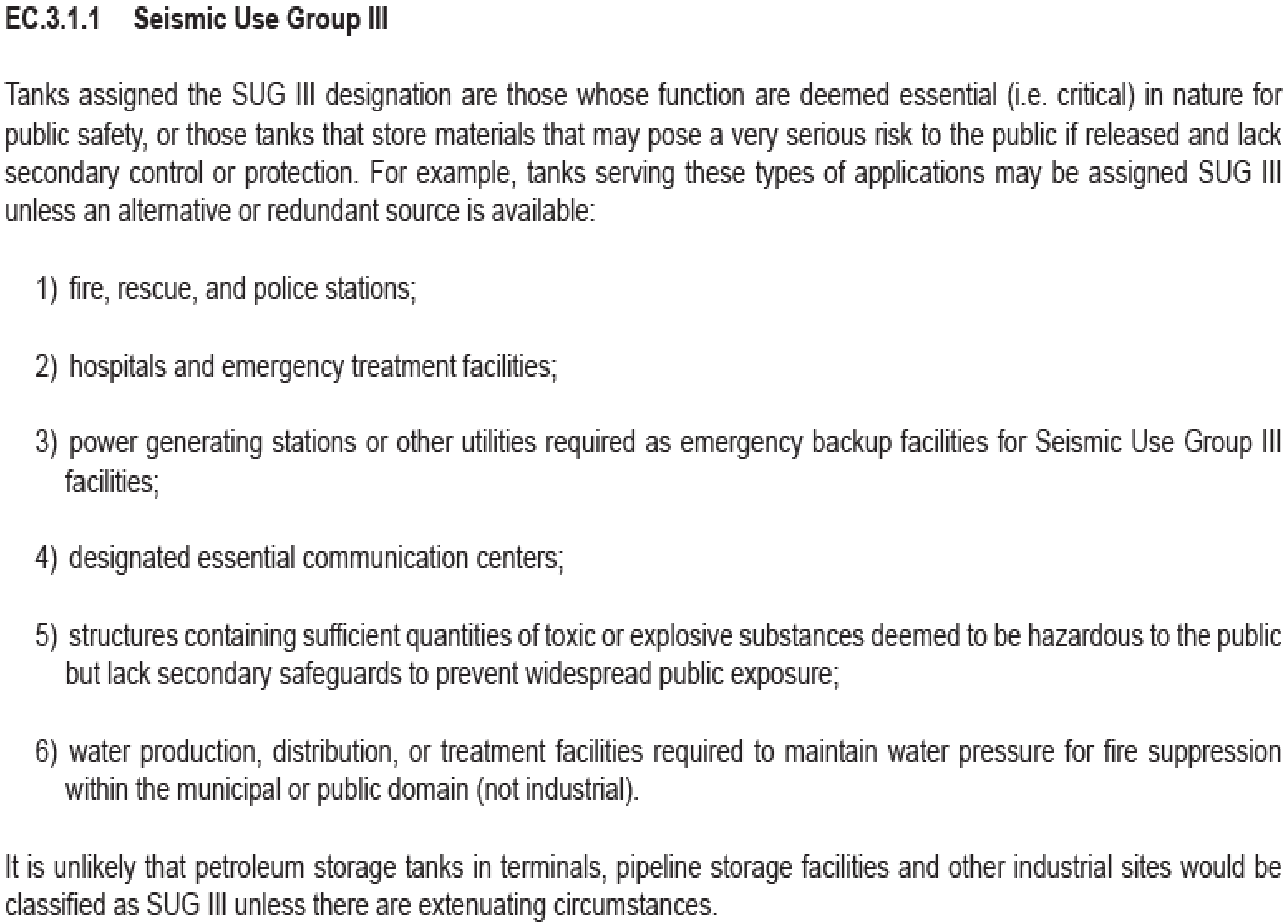
- DETERMINING SPECTRAL ACCELERATION PARAMETERS USING ASCE 7 METHOD
- DETERMINING SPECTRAL ACCELERATION PARAMETERS USING PEAK GROUND ACCELERATION
- DETERMINING SPECTRAL ACCELERATION PARAMETERS USING SITE-SPECIFIC RESPONSE SPECTRUM
- CALCULATING IMPULSIVE, CONVECTIVE AND COMBINED OVERTURNING MOMENT AND BASE SHEAR
- CALCULATING ANCHORAGE RATIO "J " AND SELF-ANCHORED ANNULAR PLATE
- CALCULATING HYDRODYNAMIC HOOP STRESSES
- CALCULATING THE OVERTURNING STABILITY RATIO

App. E (SEISMIC DESIGN OF STORAGE TANKS)



Seismic Use Group

- Seismic Use Group III
- Seismic Use Group II
- Seismic Use Group I



EC.3.1.1 Seismic Use Group III

Tanks assigned the SUG III designation are those whose function are deemed essential (i.e. critical) in nature for public safety, or those tanks that store materials that may pose a very serious risk to the public if released and lack secondary control or protection. For example, tanks serving these types of applications may be assigned SUG III unless an alternative or redundant source is available:

1) fire, rescue, and police stations;

2) hospitals and emergency treatment facilities;

3) power generating stations or other utilities required as emergency backup facilities for Seismic Use Group III facilities;

4) designated essential communication centers;

5) structures containing sufficient quantities of toxic or explosive substances deemed to be hazardous to the public but lack secondary safeguards to prevent widespread public exposure;

6) water production, distribution, or treatment facilities required to maintain water pressure for fire suppression within the municipal or public domain (not industrial).

It is unlikely that petroleum storage tanks in terminals, pipeline storage facilities and other industrial sites would be classified as SUG III unless there are extenuating circumstances.

EC.3.1.2 Seismic Use Group II

Tanks assigned the SUG II designation are those that should continue to function, after a seismic event, for public welfare, or those tanks that store materials that may pose a moderate risk to the public if released and lack secondary containment or other protection. For example, tanks serving the following types of applications may be assigned SUG II unless an alternative or redundant source is available:

1) power generating stations and other public utility facilities not included in Seismic Use Group III and required for continued operation;

2) water and wastewater treatment facilities required for primary treatment and disinfection for potable water.

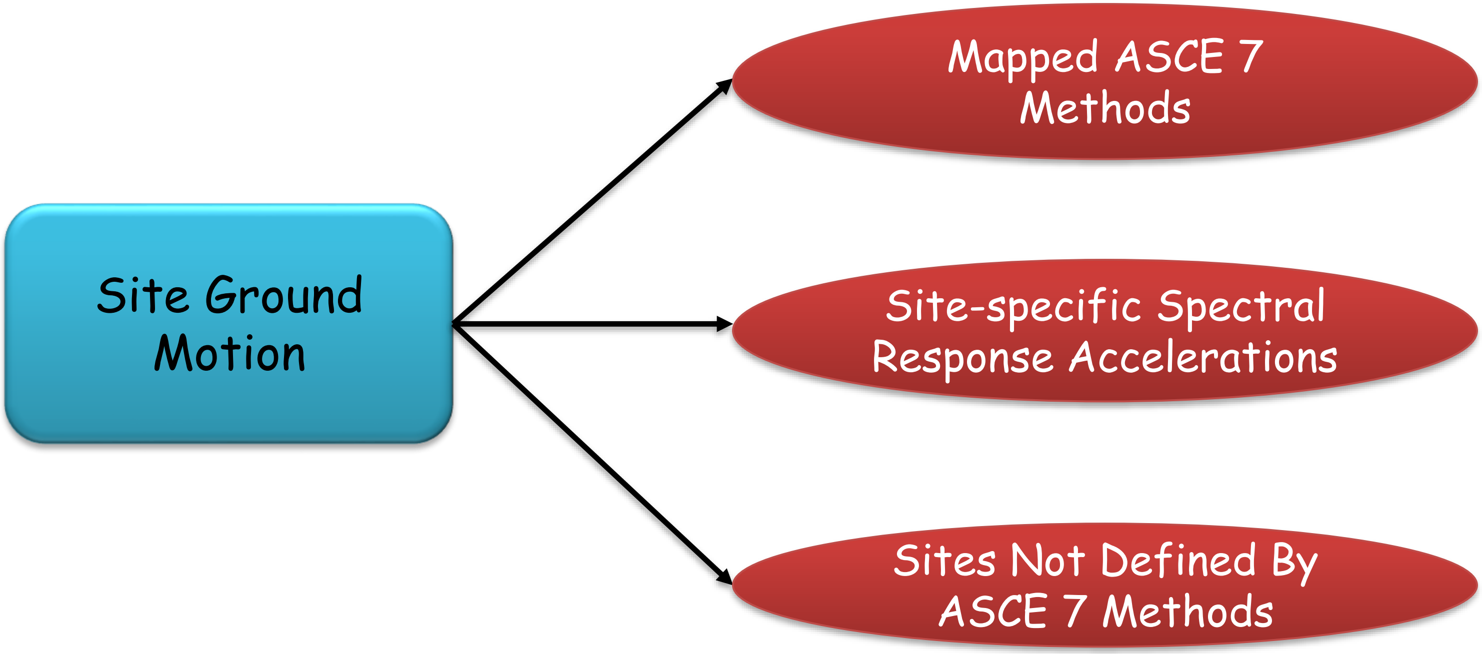
EC.3.1.3 Seismic Use Group I

SUG I is the most common classification. For example, tanks serving the following types of applications may be assigned SUG I unless an alternative or redundant source is available:

1) storage tanks in a terminal or industrial area isolated from public access that has secondary spill prevention and control;

2) storage tanks without secondary spill prevention and control systems that are sufficiently removed from areas of public access such that the hazard is minimal.

App. E (SEISMIC DESIGN OF STORAGE TANKS)



Site Ground Motion

Mapped ASCE 7 Methods

Site-specific Spectral Response Accelerations

Sites Not Defined By ASCE 7 Methods

EC.4 Site Ground Motion

The definition of the considered ground motion at the site is the first step in defining acceleration parameters and loads. The philosophy for defining the considered ground motion in the U.S. began changing about 1997. This new approach, which began with the evolution of the 1997 UBC and advanced through the efforts of the National Earthquake Hazard Reduction Program, was the basic resource for the new model building codes. Subsequent to the *International Building Code* 2000, ASCE 7 adopted the methods and is presently the basis for the US model building codes.

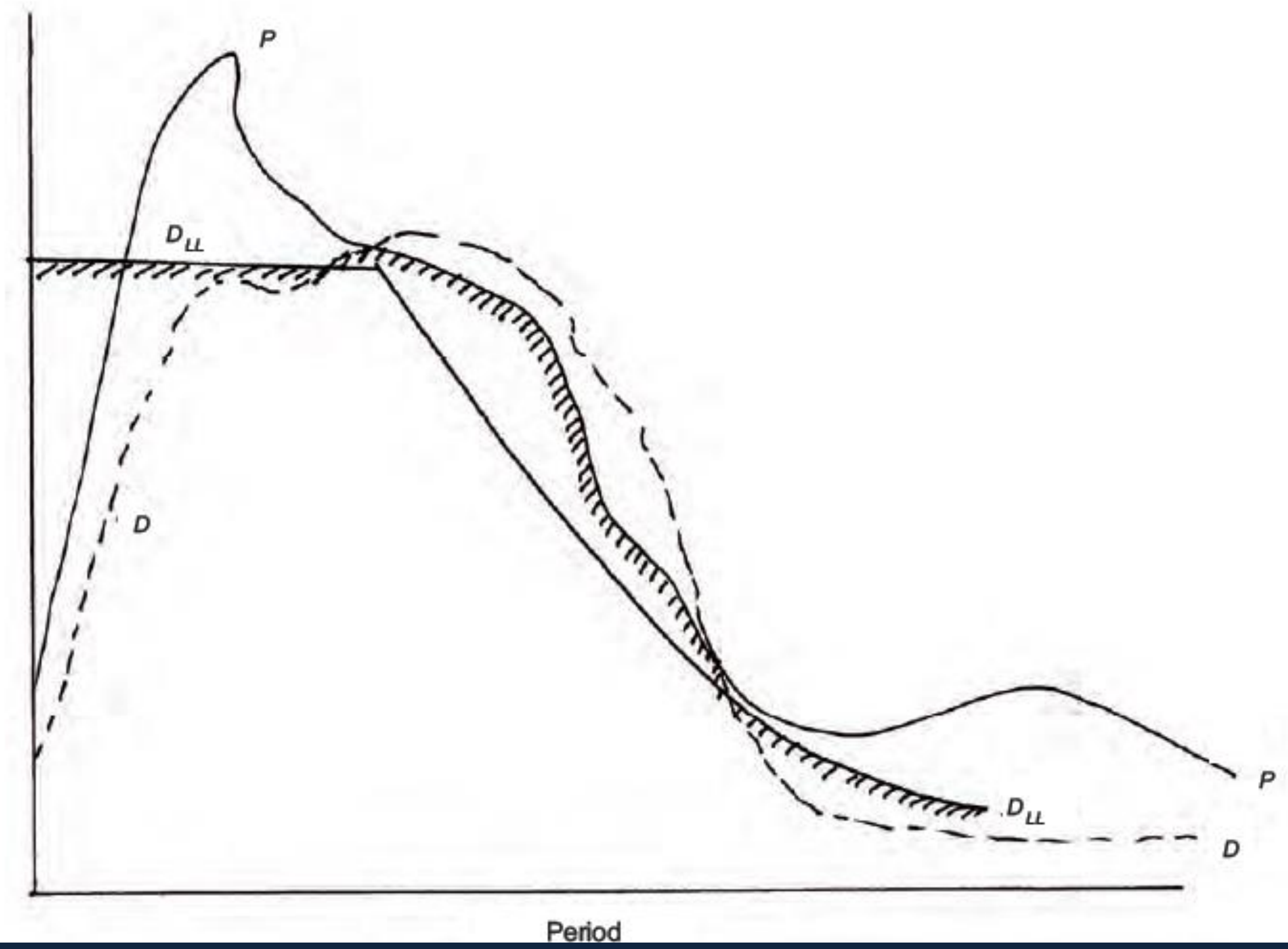
However, regulations governing seismic design for tank sites outside the U.S. may not follow this ASCE 7 approach. Therefore, this revision was written to be adaptable to these regulations. Consequently, there is no longer a definition of the "minimum" design ground motion based on US standards that applies to all sites regardless of the local regulations.

Historically, this Annex (and the U.S. standards) was based on ground motion associated with an event having a 10 % probability of exceedance in 50 years. This is an event that has a recurrence interval of 475 years. In seismically active areas where earthquakes are more frequent, such as the west coast of the US, this was a reasonable approach. In regions where earthquakes are less frequent, engineers and seismologists concluded that the hazard was under-predicted by the 475 year event. Thus, the maximum considered ground motion definition was revised to a 2 % probability of exceedance in 50 years, or a recurrence interval of about 2500 years. The economic consequences of designing to this more severe ground motion was impractical so a scaling factor was introduced based on over-strength inherently present in structures built to today's standards. See the NEHRP Provisions for a more extensive discussion of this rationale.

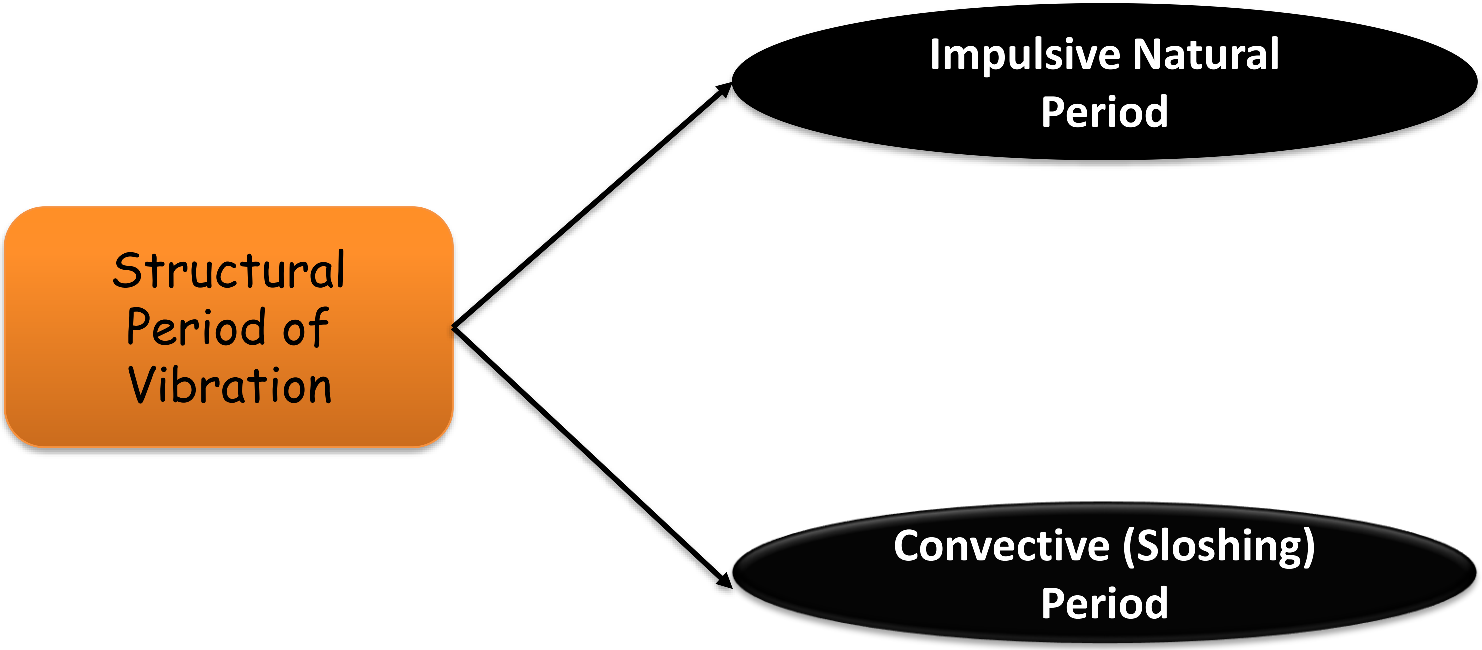
The API Seismic Task Group considered setting the 475 year event as the "minimum" for application of this standard. Given the variations worldwide in defining the ground motion, it was decided that the local regulation should set the requirements. However, the owner/specifying engineer for the tank should carefully consider the risk in selecting the appropriate design motion in areas outside the U.S. The API Seismic Task Group suggests that the 475 year event be the minimum basis for defining the site ground motion for tanks.

EC.4.2.4 Site-Specific MCE Ground Motions

Figure EC.5 illustrates conceptually how these requirements might relate to define the site specific response spectrum.



App. E (SEISMIC DESIGN OF STORAGE TANKS)



Structural Period of Vibration

Impulsive Natural Period

Convective (Sloshing) Period

EC.4.5 Structural Period of Vibration

EC.4.5.1 Impulsive Natural Period

To use the methods in this Annex, the impulsive seismic acceleration parameter is independent of tank system period unless a site-specific analysis or soil structure interaction evaluation is performed. The impulsive period of the tank is nearly always less than T_S, placing it on the plateau of the response spectra. Thus, the impulsive acceleration parameter is based directly on S_{DS}. For special circumstances, a simplified procedure was included in the Annex to 14
determine the impulsive period which was taken from the following reference:

> "Simplified Procedure for Seismic Analysis of Liquid-Storage Tanks," Malhotra, P; Wenk, T; and Wieland, M. *Structural Engineering International*, March 2000.

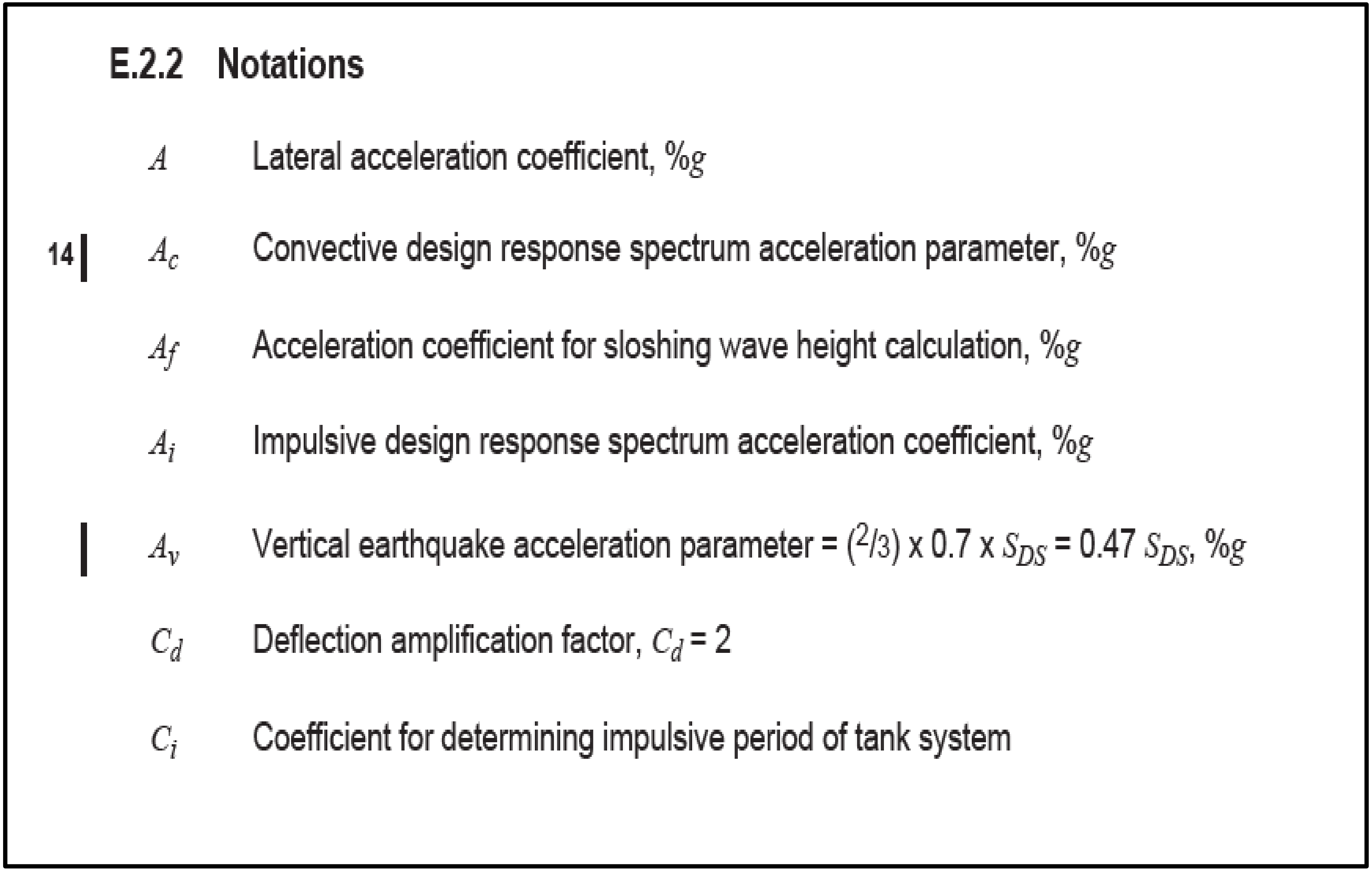
EC.4.5.2 Convective (Sloshing) Period

For convenience, the graphical procedure for determining the sloshing period, T_c, is included here. See Equation E.4.5.2-b and Figure EC.5.

$$T_c = K_s\sqrt{D} \qquad \text{(E.4.5.2-b)}$$

where

D is the nominal tank diameter in ft;



E.2.2 Notations

	A	Lateral acceleration coefficient, %g
14	A_c	Convective design response spectrum acceleration parameter, %g
	A_f	Acceleration coefficient for sloshing wave height calculation, %g
	A_i	Impulsive design response spectrum acceleration coefficient, %g
	A_v	Vertical earthquake acceleration parameter = ($^2/3$) x 0.7 x S_{DS} = 0.47 S_{DS}, %g
	C_d	Deflection amplification factor, C_d = 2
	C_i	Coefficient for determining impulsive period of tank system

App. E (SEISMIC DESIGN OF STORAGE TANKS)

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

where

$$V_i = A_i(W_s + W_r + W_f + W_i)$$

$$V_c = A_c W_c$$

App. E (SEISMIC DESIGN OF STORAGE TANKS)

Impulsive spectral acceleration parameter, A_i:

$$A_i = S_{DS}\left(\frac{I}{R_{wi}}\right) = 2.5QF_aS_0\left(\frac{I}{R_{wi}}\right) \qquad \text{(E.4.6.1-1)}$$

However, $A_i \geq 0.007$ (E.4.6.1-2)

and, for $S_1 \geq 0.6$:

$$A_i \geq 0.5S_1\left(\frac{I}{R_{wi}}\right) = 0.625S_P\left(\frac{I}{R_{wi}}\right) \qquad \text{(E.4.6.1-3)}$$

Convective spectral acceleration parameter, A_c:

When, $T_C \leq T_L$

$$A_c = KS_{D1}\left(\frac{1}{T_c}\right)\left(\frac{I}{R_{wc}}\right) = 2.5KQF_aS_0\left(\frac{T_s}{T_c}\right)\left(\frac{I}{R_{wc}}\right) \leq A_i \qquad \text{(E.4.6.1-4)}$$

When, $T_C > T_L$

$$A_c = KS_{D1}\left(\frac{T_L}{T_c^2}\right)\left(\frac{I}{R_{wc}}\right) = 2.5KQF_aS_0\left(\frac{T_sT_L}{T_c^2}\right)\left(\frac{I}{R_{wc}}\right) \leq A_i \qquad \text{(E.4.6.1-5)}$$

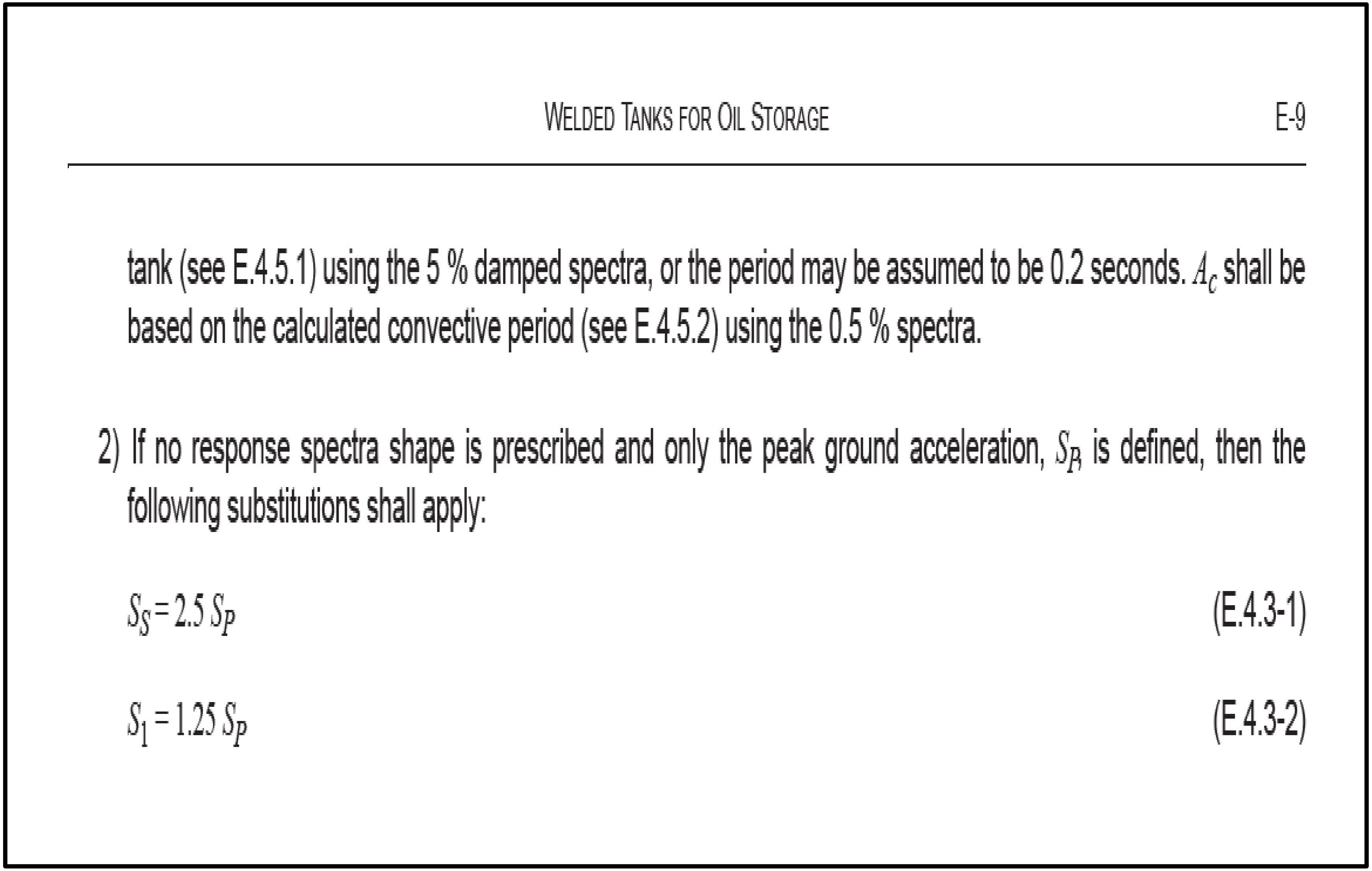
App. E (SEISMIC DESIGN OF STORAGE TANKS)

- SDs
- I
- Rwi
- Rwc
- Q
- Fa
- Fv
- S0
- SD1
- Ts
- Tc
- K
- TL

App. E (SEISMIC DESIGN OF STORAGE TANKS)

S_0	Mapped, maximum considered earthquake, 5 % damped, spectral response acceleration parameter at a period of zero seconds (peak ground acceleration for a rigid structure), $\%g$
S_1	Mapped, maximum considered earthquake, 5 % damped, spectral response acceleration parameter at a period of one second, $\%g$
S_a	The 5 % damped, design spectral response acceleration parameter at any period based on mapped, probabilistic procedures, $\%g$
S_a^*	The 5 % damped, design spectral response acceleration parameter at any period based on site-specific procedures, $\%g$
S_{a0}^*	The 5 % damped, design spectral response acceleration parameter at zero period based on site-specific procedures, $\%g$
S_{D1}	The design, 5 % damped, spectral response acceleration parameter at one second based on the ASCE 7 methods, equals $Q\,F_v\,S_1$, $\%g$
S_{DS}	The design, 5% damped, spectral response acceleration parameter at short periods (T = 0.2 seconds) based on ASCE 7 methods, equals $Q\,F_a\,S_s$, $\%g$
S_P	Design level peak ground acceleration parameter for sites not addressed by ASCE methods. [See EC Example Problem 2 when using "Z" factor from earlier editions of API 650 and UBC. Since 475 year recurrence interval is basis of this peak ground acceleration, Q = 1.0 (no scaling).]
S_S	Mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at short periods (0.2 sec), $\%g$
s_u	Undrained shear strength, ASTM D2166 or ASTM D2850

App. E (SEISMIC DESIGN OF STORAGE TANKS)



tank (see E.4.5.1) using the 5 % damped spectra, or the period may be assumed to be 0.2 seconds. A_c shall be based on the calculated convective period (see E.4.5.2) using the 0.5 % spectra.

2) If no response spectra shape is prescribed and only the peak ground acceleration, S_P, is defined, then the following substitutions shall apply:

$$S_S = 2.5\, S_P \tag{E.4.3-1}$$

$$S_1 = 1.25\, S_P \tag{E.4.3-2}$$

App. E (SEISMIC DESIGN OF STORAGE TANKS)

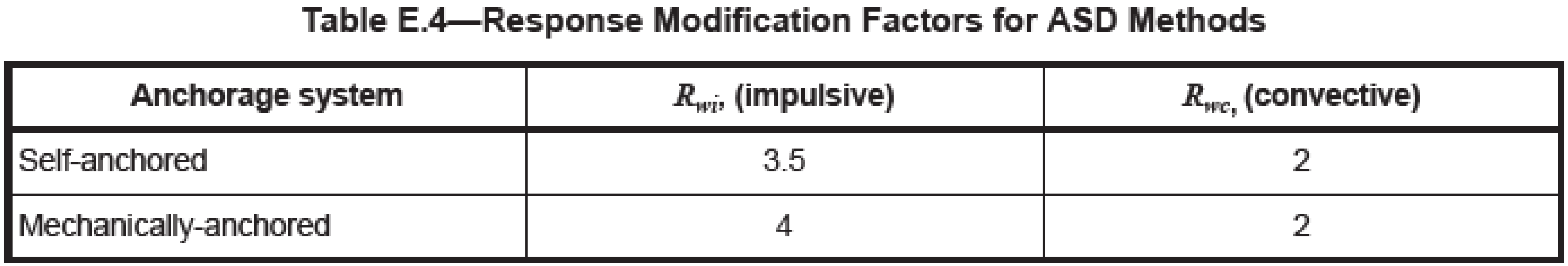


Table E.4—Response Modification Factors for ASD Methods

Anchorage system	R_{wi}, (impulsive)	R_{wc}, (convective)
Self-anchored	3.5	2
Mechanically-anchored	4	2

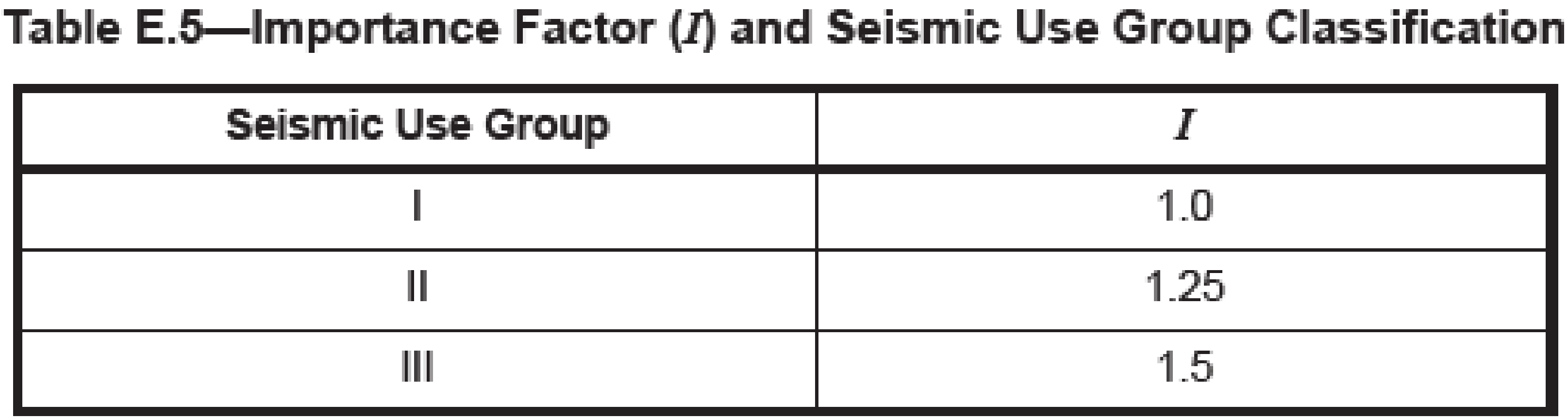


Table E.5—Importance Factor (I) and Seismic Use Group Classification

Seismic Use Group	I
I	1.0
II	1.25
III	1.5

App. E (SEISMIC DESIGN OF STORAGE TANKS)

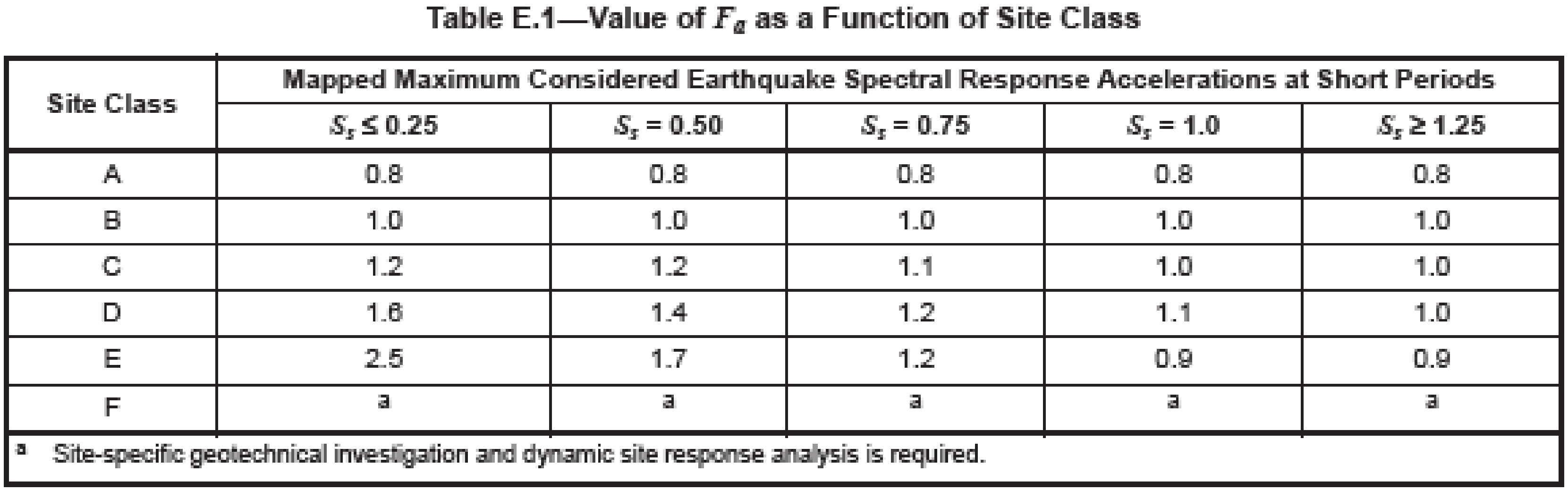


Table E.1—Value of F_a as a Function of Site Class

Site Class	Mapped Maximum Considered Earthquake Spectral Response Accelerations at Short Periods				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.0$	$S_s \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	a	a	a	a	a

[a] Site-specific geotechnical investigation and dynamic site response analysis is required.

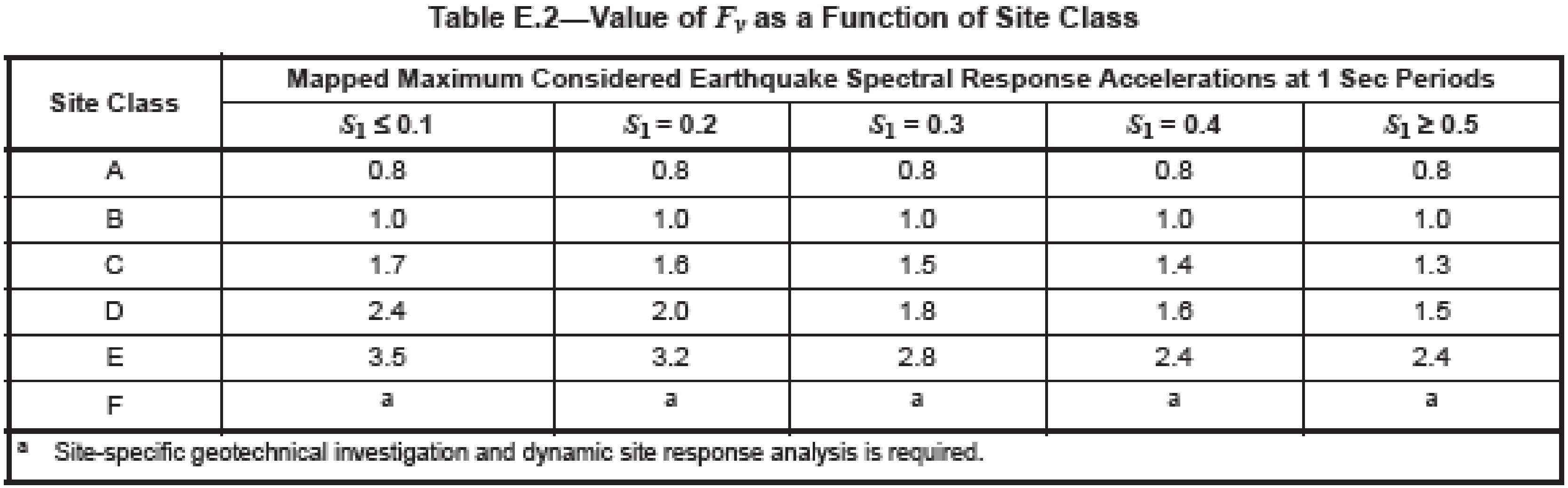
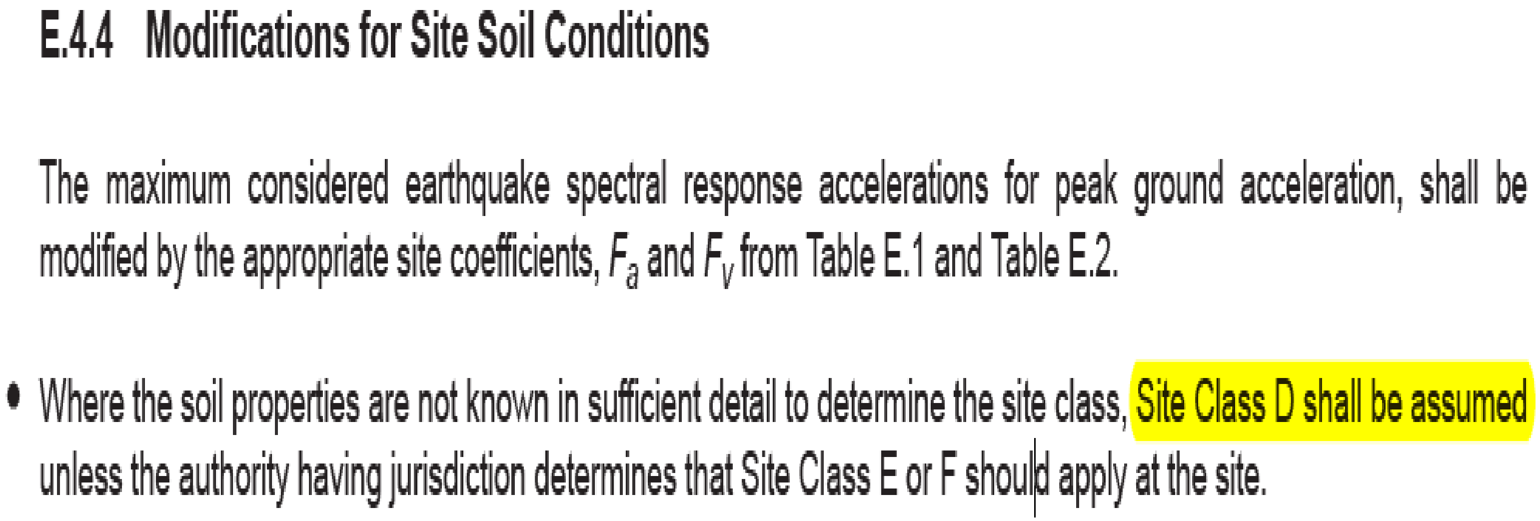


Table E.2—Value of F_v as a Function of Site Class

Site Class	Mapped Maximum Considered Earthquake Spectral Response Accelerations at 1 Sec Periods				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a	a	a	a	a

[a] Site-specific geotechnical investigation and dynamic site response analysis is required.

App. E (SEISMIC DESIGN OF STORAGE TANKS)



E.4.4 Modifications for Site Soil Conditions

The maximum considered earthquake spectral response accelerations for peak ground acceleration, shall be modified by the appropriate site coefficients, F_a and F_v from Table E.1 and Table E.2.

- Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be assumed unless the authority having jurisdiction determines that Site Class E or F should apply at the site.

App. E (SEISMIC DESIGN OF STORAGE TANKS)

- ✓ SDs
- ✓ I
- ✓ Rwi
- ✓ Rwc
- ✓ Q
- ✓ Fa
- ✓ Fv
- ✓ S0
- ✓ SD1
- ☐ Ts
- ☐ Tc
- ☐ K (1.5 unless otherwise specified)
- ☐ Ks
- ☐ TL

T0 : 0.2 *FvS1 / FaSS*

TS : FvS1 / FaSS

$$T_c = 1.8K_s\sqrt{D} \qquad \text{(E.4.5.2-a)}$$

or, in USC units:

$$T_c = K_s\sqrt{D} \qquad \text{(E.4.5.2-b)}$$

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68H}{D}\right)}} \qquad \text{(E.4.5.2-c)}$$

App. E (SEISMIC DESIGN OF STORAGE TANKS)

- ✓ Ws : Total weight of tank shell and appurtenances, N (lbf)
- ✓ Wr : Total weight of fixed tank roof including framing, knuckles, any permanent attachments and 10 % of the roof balanced design snow load, Sb, N (lbf)
- ✓ Wf : Weight of the tank bottom, N
- ✓ Wi : Effective impulsive portion of the liquid weight, N
- ✓ Wc : Effective convective (sloshing) portion of the liquid weight, N

E.6.1.1 Effective Weight of Product

The effective weights W_i and W_c shall be determined by multiplying the total product weight, W_p, by the ratios W_i/W_p and W_c/W_p, respectively, Equations E.6.1.1-1 through E.6.1.1-3.

When D/H is greater than or equal to 1.333, the effective impulsive weight is defined in Equation E.6.1.1-1:

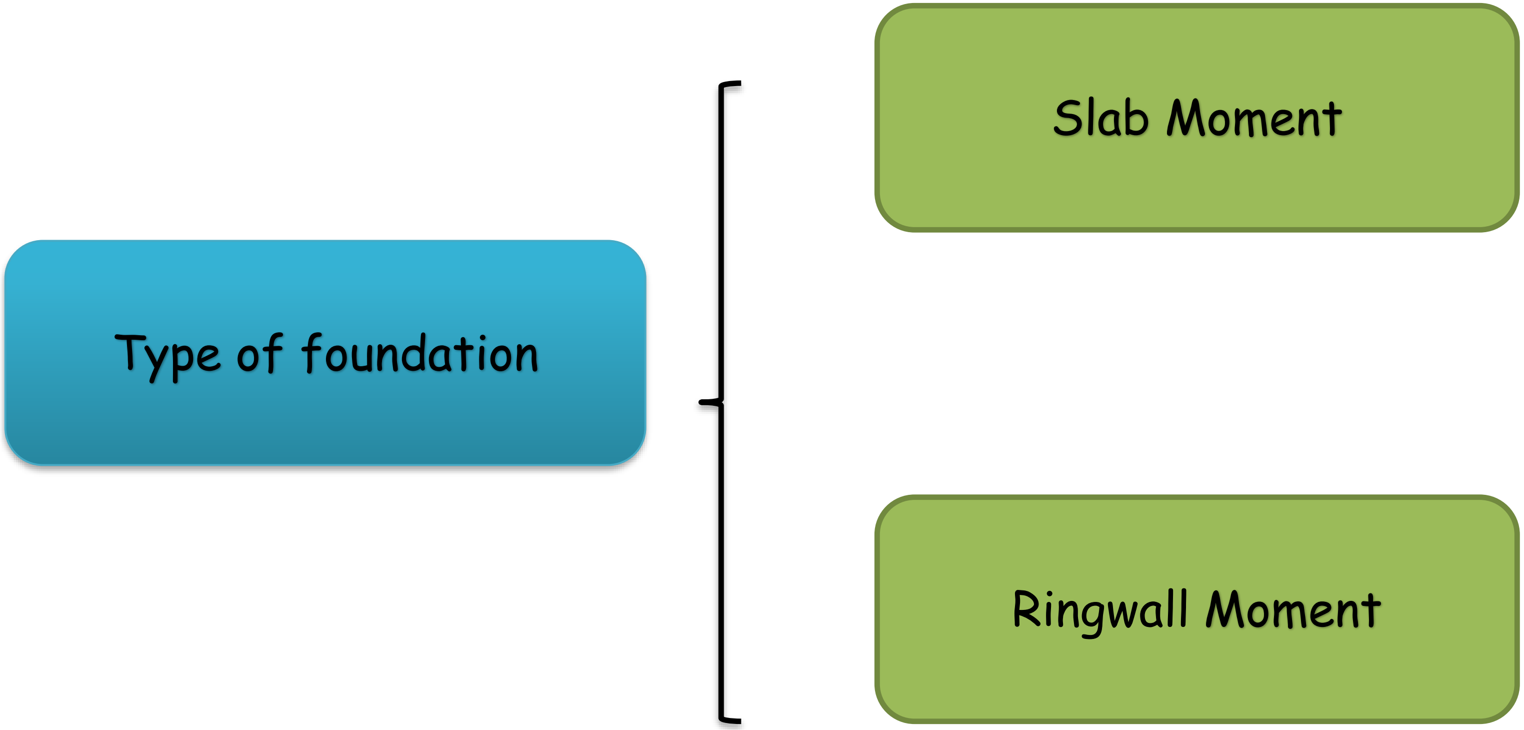
$$W_i = \frac{\tanh\left(0.866\frac{D}{H}\right)}{0.866\frac{D}{H}} W_p \qquad \text{(E.6.1.1-1)}$$

When D/H is less than 1.333, the effective impulsive weight is defined in Equation E.6.1.1-2:

$$W_i = \left[1.0 - 0.218\frac{D}{H}\right] W_p \qquad \text{(E.6.1.1-2)}$$

The effective convective weight is defined in Equation E.6.1.1-3:

$$W_c = 0.230\frac{D}{H}\tanh\left(\frac{3.67H}{D}\right) W_p \qquad \text{(E.6.1.1-3)}$$



Type of foundation

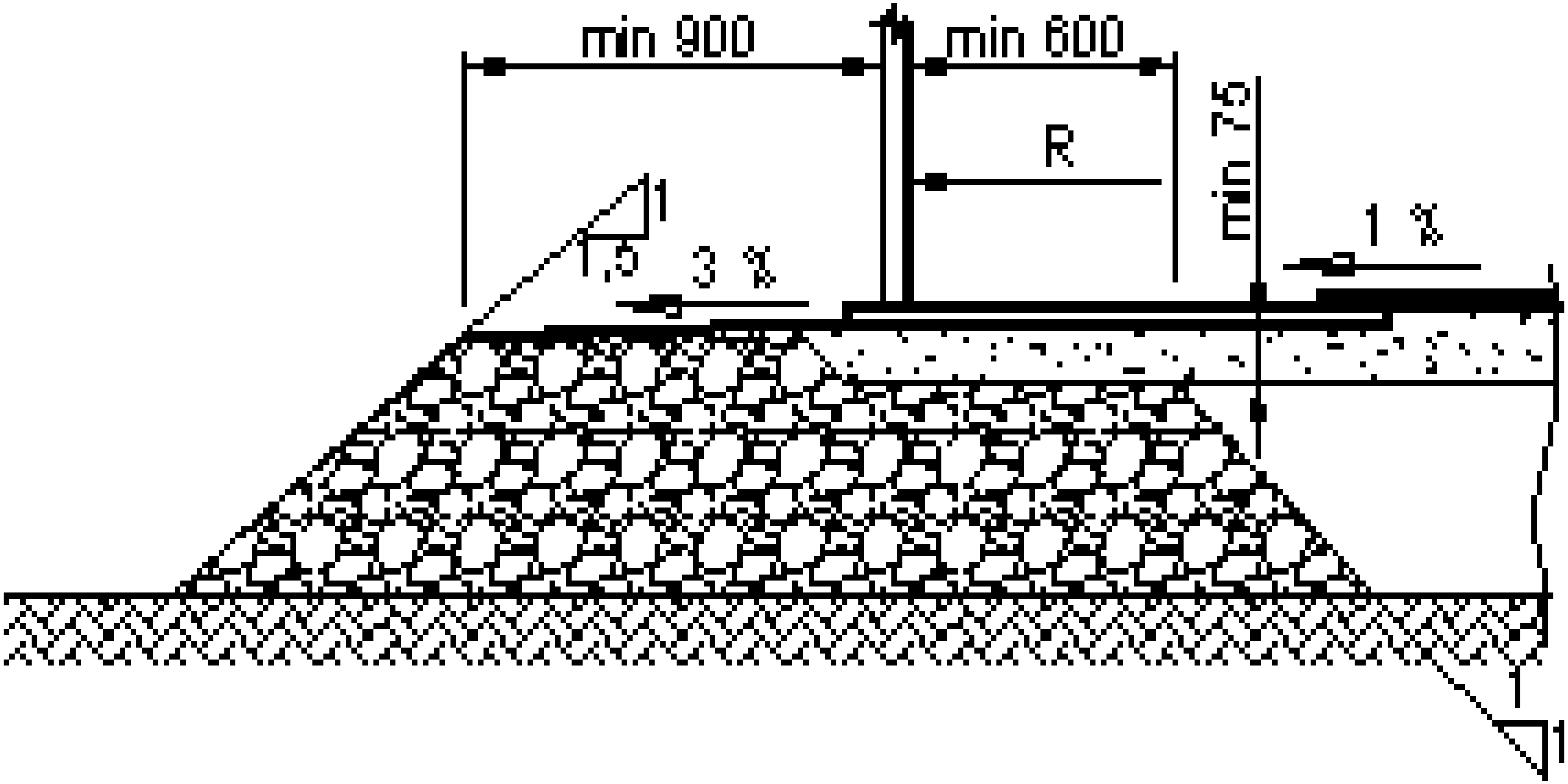
Slab Moment

Ringwall Moment

Type of foundation construction

a) foundation from compacted soil (earth type foundation). This is most often applied scheme of foundation construction because it is cheapest and easiest for execution. It is made according to the scheme shown in standard API 650. It is used when the soil can bear the pressure of the upper steel construction and when the anchorage is not necessary. Even when there is small leak moving outof the soil is possible. It can leads to destruction of the tank. In this reason the diameter of the tank must be bigger than the diameter of the tank with not less then 1,8 m. Earth type foundation does not allow good leveling of the bottom i.e. of the shell of the tank. When they are used it is possible the uneven settlement which cause additional efforts in the tank's elements.

ref.: http://www.astanks.com



Earth type foundation

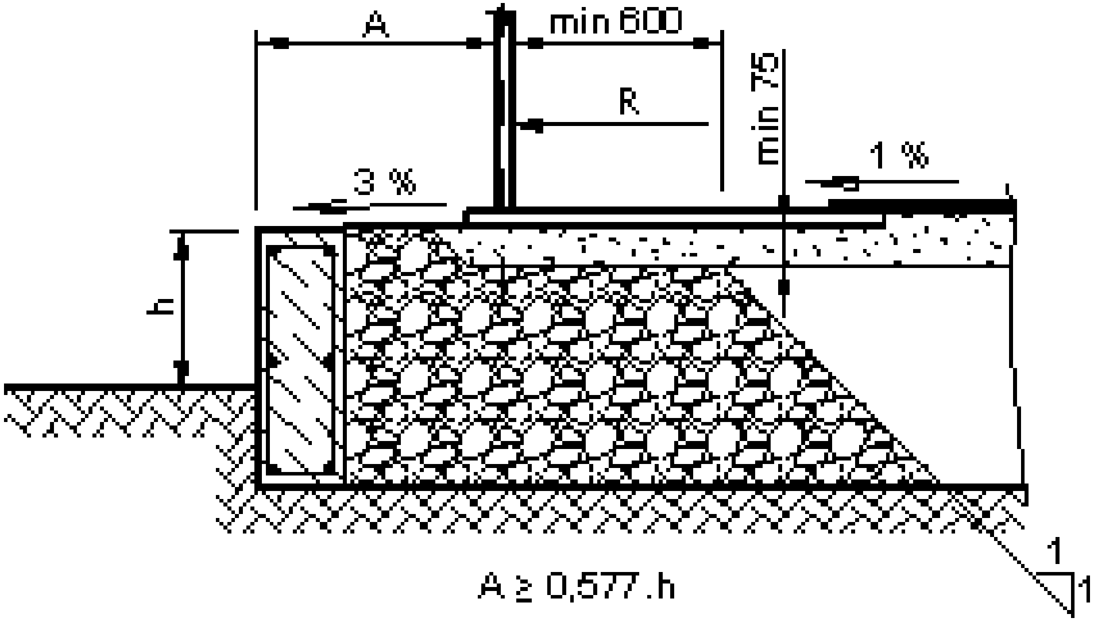
Type of foundation construction

b) reinforced concrete ring which is not placed under the shell.

The tanks which are subject of this research are the above ground facilities. They are placed on 0,3 ÷1,2 m above the soil. This level difference is remarkable in the fuel oil tanks where pump always must be under the liquids ($\Delta h \geq 0{,}7$m). If the classical earth type foundation reaches this height the facility must occupy remarkable surface on the site. The reinforcing of such different leveled surfaces bears a risk for landslide (when the earth is covered by grass or asphalt) or it is slow, expensive and work consuming process (when the earth is paved).

In order to avoid this inconvenience appears the idea of small foundation ring between the ground and the bottom level which ring is a combination free sand pillow and reinforced concrete ring in the periphery. The proposed construction is similar to the API Std. 650, but the foundation ring is moved in the outside direction where it can not be influenced of the load of the shell and the tank roof upon it. When there is soil settlement under the tank, the reinforced concrete ring does not allow full drain of the water so that this solution is unfortunate. It should not be applied to the new build tanks.

ref.: http://www.astanks.com

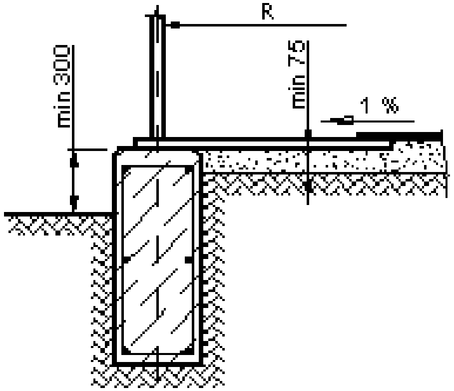


Reinforced concrete ringwall foundation which is not placed under the shell

Type of foundation construction

c) reinforced concrete ring wall foundation.

The trend in the tanks building shows that the volumes of the facility increase. Spatial steel construction of the tanks stands more flexible. In this reason bigger attention must be paid to the shell settlement and to the prevention measures. The use of the rigid reinforced concrete ring increase around the world. When the tanks are bigger the dimensions of the rings are: largeness not less then 0,6 m and height 1,5 – 2,0 m.This type foundation construction allows very good leveling of the periphery of the bottom and the shell which is positioned on it. The uneven settlement of the tank is limited. It is possible anchors to be put there.



Reinforced concrete ringwall foundation

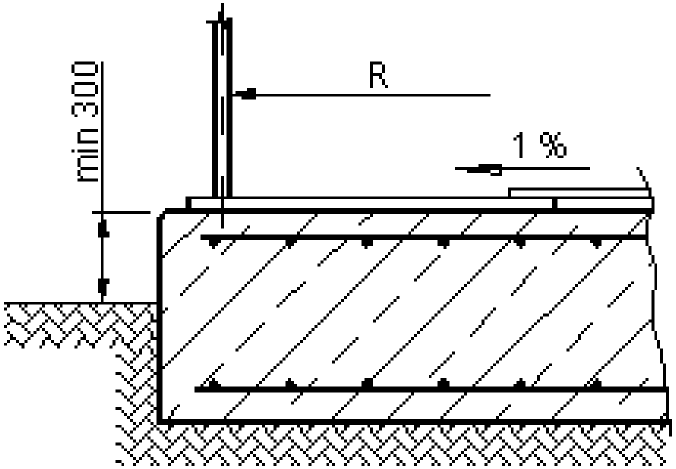
ref.: http://www.astanks.com

Type of foundation construction

d) reinforced concrete slab

They could be applied when the tanks are relatively small because this type of foundations is very expensive.

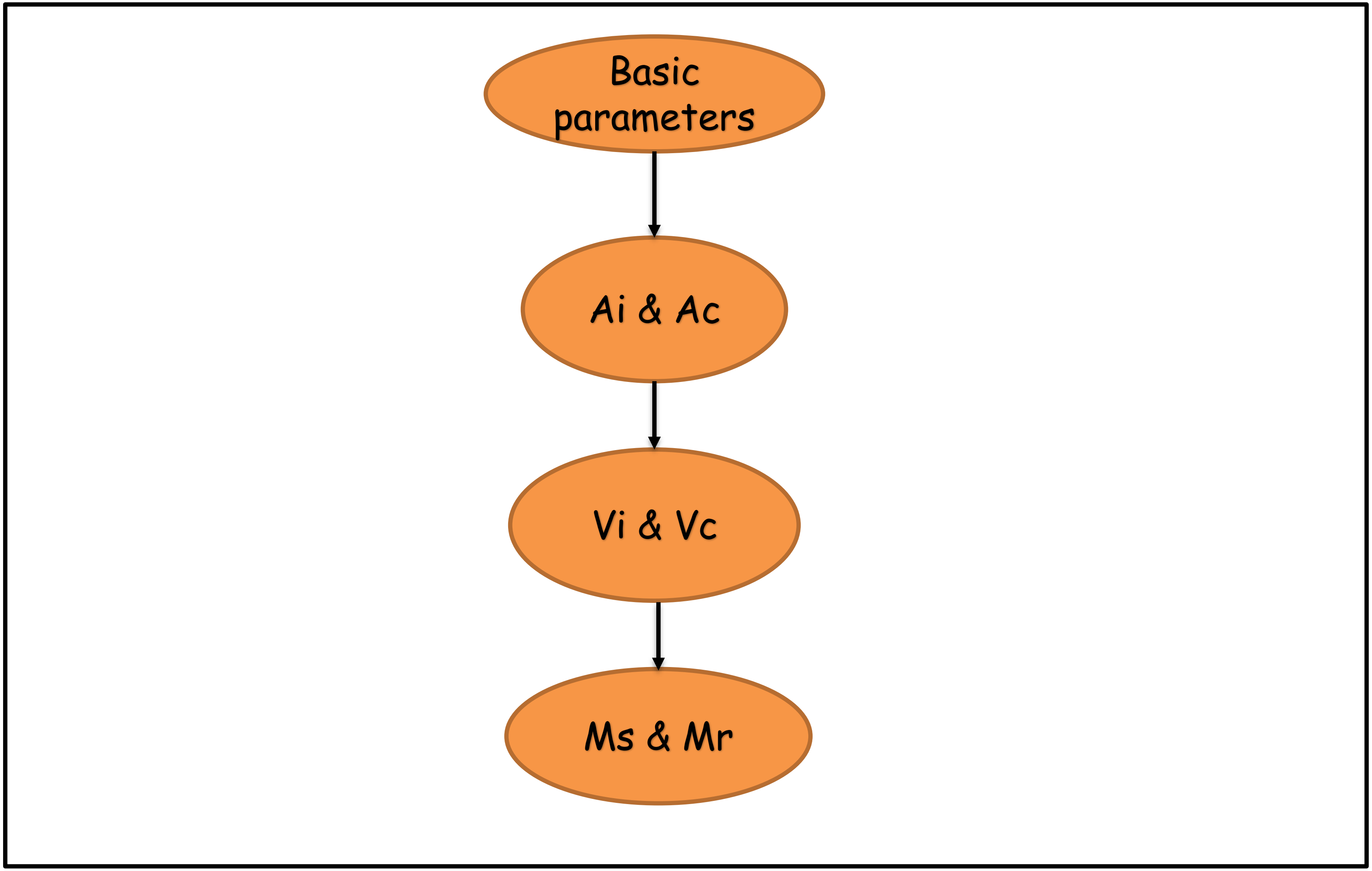
Thick concrete slabs are more favorable for upper steel structure. They do not allow the uneven settlement of the tank. The reinforced concrete slabs are very recommendable when the level of the underground water is high.



Reinforced concrete slab

ref.: http://www.astanks.com

App. E (SEISMIC DESIGN OF STORAGE TANKS)



Basic parameters

Ai & Ac

Vi & Vc

Ms & Mr

App. E (SEISMIC DESIGN OF STORAGE TANKS)

Ringwall Moment, M_{rw}:

$$M_{rw} = \sqrt{[A_i(W_iX_i + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_c)]^2}$$

Slab Moment, M_S:

$$M_s = \sqrt{[A_i(W_iX_{is} + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_{cs})]^2}$$

App. E (SEISMIC DESIGN OF STORAGE TANKS)

❑Xi (E.6.1.2.1-1) & (E.6.1.2.1-2)	If D/H >= 1.33 $X_i = 0.375H$ If D/H < 1.33 $X_i = \left[0.5 - 0.094\frac{D}{H}\right]H$
❑Xs	$X_c = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H$
❑Xr	Height from the bottom of the tank shell to the roof and roof appurtenances center of gravity, m
❑Xc (E.6.1.2.1-3)	$X_c = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H$

App. E (SEISMIC DESIGN OF STORAGE TANKS)

❑Xis (E.6.1.2.2-1) & (E.6.1.2.2-2)	If D/H >= 1.33 If D/H < 1.33	$X_{is} = 0.375\left[1.0 + 1.333\left(\frac{0.866\frac{D}{H}}{\tanh\left(0.866\frac{D}{H}\right)} - 1.0\right)\right]H$ $X_{is} = \left[0.500 + 0.060\frac{D}{H}\right]H$
❑Xcs (E.6.1.2.2-3)		$X_{cs} = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1.937}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H$

App. E (Anchorage Ratio, J)

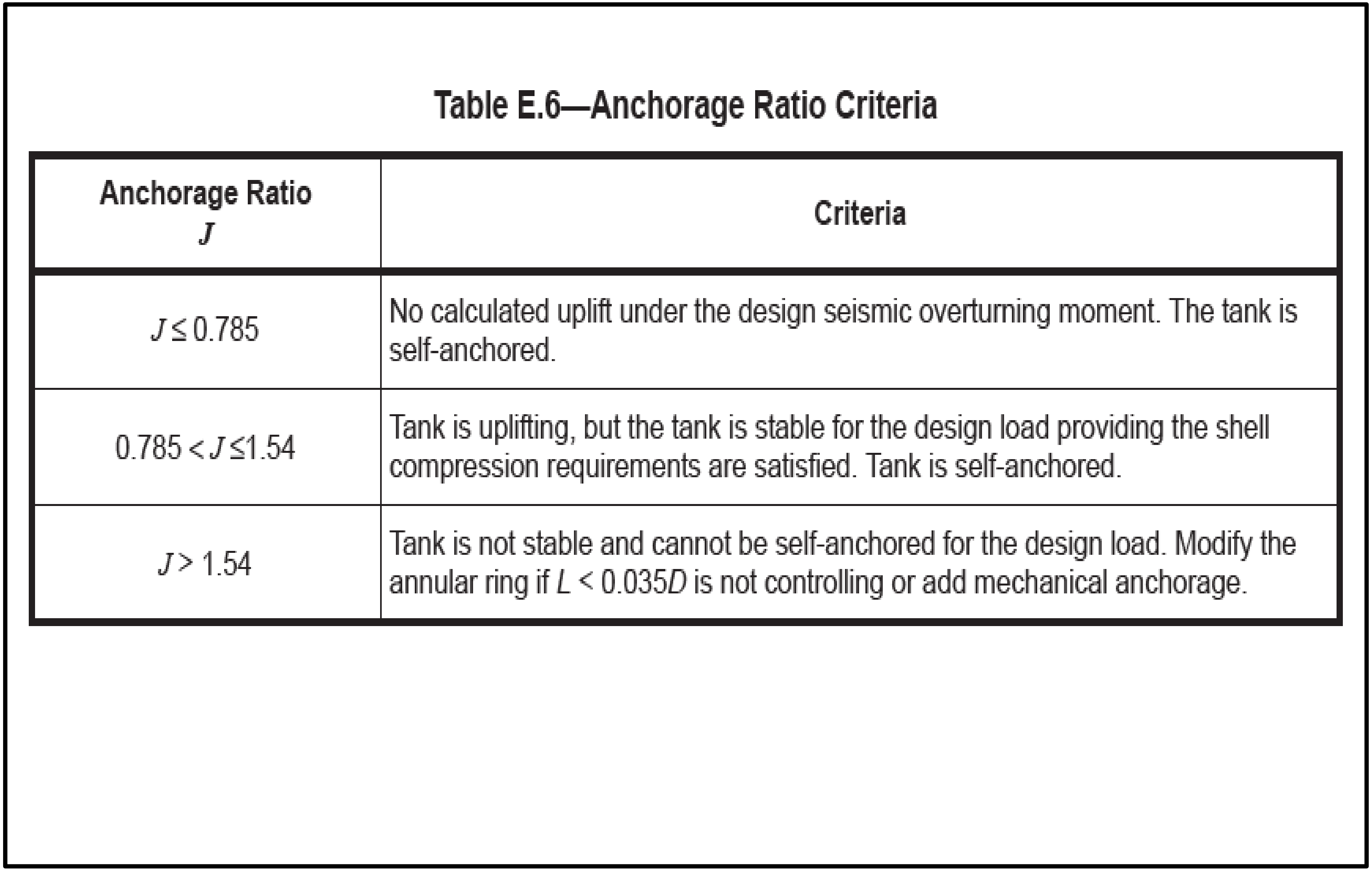


Table E.6—Anchorage Ratio Criteria

Anchorage Ratio J	Criteria
$J \leq 0.785$	No calculated uplift under the design seismic overturning moment. The tank is self-anchored.
$0.785 < J \leq 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and cannot be self-anchored for the design load. Modify the annular ring if $L < 0.035D$ is not controlling or add mechanical anchorage.

App. E (Anchorage Ratio, J)

$$J = \frac{M_{rw}}{D^2[w_t(1 - 0.4A_v) + w_a - 0.4w_{int}]}$$

where

$$w_t = \left[\frac{W_s}{\pi D} + w_{rs}\right]$$

App. E (Anchorage Ratio, J)

Av = Vertical earthquake acceleration parameter = (2/3) x 0.7 x SDS = 0.47 SDS, %g.

The vertical seismic acceleration parameter shall be taken as 0.47SDS, unless otherwise specified by the Purchaser. Alternatively, the Purchaser may specify the vertical ground motion acceleration. That acceleration shall be multiplied by 0.7 to obtain the vertical acceleration parameter, Av.

Wt=Tank and roof weight acting at base of shell, N/m

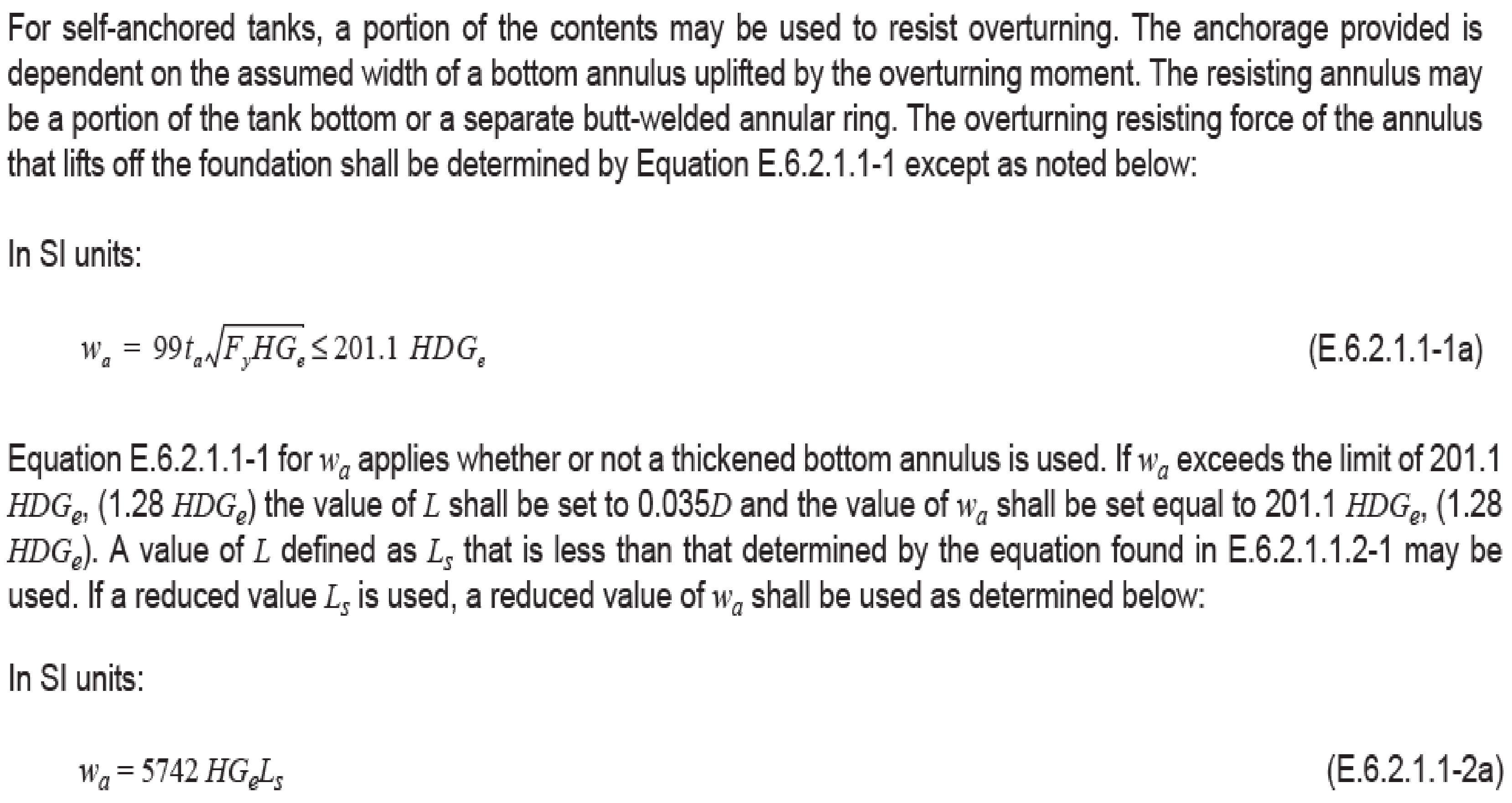
Wint= Calculated design uplift load due to product pressure per unit circumferential length, N/m

Wrs= Roof load acting on the shell, including 10 % of the roof balanced design snow load, Sb, N/m

Ws=Total weight of tank shell and appurtenances, N

App. E (Anchorage Ratio, J)

Wa = Force resisting uplift in annular region, N/m



For self-anchored tanks, a portion of the contents may be used to resist overturning. The anchorage provided is dependent on the assumed width of a bottom annulus uplifted by the overturning moment. The resisting annulus may be a portion of the tank bottom or a separate butt-welded annular ring. The overturning resisting force of the annulus that lifts off the foundation shall be determined by Equation E.6.2.1.1-1 except as noted below:

In SI units:

$$w_a = 99 t_a \sqrt{F_y H G_e} \leq 201.1\ H D G_e \qquad \text{(E.6.2.1.1-1a)}$$

Equation E.6.2.1.1-1 for w_a applies whether or not a thickened bottom annulus is used. If w_a exceeds the limit of 201.1 HDG_e, (1.28 HDG_e) the value of L shall be set to 0.035D and the value of w_a shall be set equal to 201.1 HDG_e, (1.28 HDG_e). A value of L defined as L_s that is less than that determined by the equation found in E.6.2.1.1.2-1 may be used. If a reduced value L_s is used, a reduced value of w_a shall be used as determined below:

In SI units:

$$w_a = 5742\ H G_e L_s \qquad \text{(E.6.2.1.1-2a)}$$

App. E (Maximum Longitudinal Shell-Membrane Compression Stress)

E.6.2.2.1 Shell Compression in Self-Anchored Tanks

The maximum longitudinal shell compression stress at the bottom of the shell when there is no calculated uplift, $J < 0.785$, shall be determined by the formula:

In SI units:

$$\sigma_c = \left(w_t(1 + 0.4A_v) + \frac{1.273M_{rw}}{D^2} \right) \frac{1}{1000t_s} \qquad \text{(E.6.2.2.1-1a)}$$

The maximum longitudinal shell compression stress at the bottom of the shell when there is calculated uplift, $J > 0.785$, shall be determined by the formula:

In SI units:

$$\sigma_c = \left(\frac{w_t(1 + 0.4A_v) + w_a}{0.607 - 0.18667[J]^{2.3}} - w_a \right) \frac{1}{1000t_s} \qquad \text{(E.6.2.2.1-2a)}$$

App. E (Maximum Longitudinal Shell-Membrane Compression Stress)

E.6.2.2.2 Shell Compression in Mechanically-Anchored Tanks

- The maximum longitudinal shell compression stress at the bottom of the shell for mechanically-anchored tanks shall be determined by the formula:

$$\sigma_c = \left(w_t(1 + 0.4A_v) + \frac{1.273 M_{rw}}{D^2} \right) \frac{1}{1000 t_s}$$

App. E (Maximum Longitudinal Shell-Membrane Compression Stress)

E.6.2.2.3 Allowable Longitudinal Shell-Membrane Compression Stress in Tank Shell

When GHD^2/t^2 is ≥ 44 (SI units) (10^6 USC units),

In SI units:

$$F_C = 83\ t_s/D \tag{E.6.2.2.3-1a}$$

or, in USC units:

$$F_C = 10^6\ t_s/D \tag{E.6.2.2.3-1b}$$

In SI units:

When GHD^2/t^2 is < 44:

$$F_C = 83t_s/(2.5D) + 7.5\sqrt{(GH)} < 0.5F_{ty} \tag{E.6.2.2.3-2a}$$

or, in USC units:

When GHD^2/t^2 is less than 1×10^6:

$$F_C = 10^6\ t_s/(2.5D) + 600\sqrt{(GH)} < 0.5F_{ty} \tag{E.6.2.2.3-2b}$$

App. E (Dynamic Liquid Hoop Forces)

Dynamic hoop tensile stresses due to the seismic motion of the liquid shall be determined by the following formulas:

$$\sigma_T = \sigma_h \pm \sigma_s = \frac{N_h \pm \sqrt{N_i^2 + N_c^2}}{t}$$

When vertical acceleration is specified.

$$\sigma_T = \sigma_h \pm \sigma_s = \frac{N_h \pm \sqrt{N_i^2 + N_c^2 + (A_v N_h / 2.5)^2}}{t}$$

App. E (Dynamic Liquid Hoop Forces)

Dynamic hoop tensile stresses due to the seismic motion of the liquid shall be determined by the following formulas:

For D/H ≥ 1.333:

$$N_i = 8.48A_iGDH\left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right]\tanh\left(0.866\frac{D}{H}\right)$$

For D/H < 1.33 and Y < 0.75D:

$$N_i = 5.22A_iGD^2\left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right]$$

For D/H < 1.333 and Y > 0.75D:

$$N_i = 2.6A_iGD^2$$

App. E (Dynamic Liquid Hoop Forces)

For all proportions of *D/H*:

In SI units:

$$N_c = \frac{1.85 A_c G D^2 \cosh\left[\frac{3.68(H-Y)}{D}\right]}{\cosh\left[\frac{3.68H}{D}\right]}$$

App. E (Maximum Hoop Stress)

The maximum allowable hoop tension membrane stress for the combination of hydrostatic product and dynamic membrane hoop effects shall be the lesser of:

- The basic allowable membrane in this standard for the shell plate material increased by 33 %; or
- 0.9Fy times the joint efficiency where Fy is the lesser of the published minimum yield strength of the shell material or weld material.

App. E (Freeboard E.7.2)

- Freeboard is required for SUG II and SUG III tanks. The height of the sloshing wave above the product design height can be estimated by:

$\delta_s = 0.42\,DA_f$ (see Note c in Table E.7)

For SUG I and II,

When, $T_C \le 4$ $\quad A_f = KS_{D1}I\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0I\left(\frac{T_S}{T_C}\right)$

When, $T_C > 4$ $\quad A_f = KS_{D1}I\left(\frac{4}{T_C^2}\right) = 2.5KQF_aS_0I\left(\frac{4T_S}{T_C^2}\right)$

For SUG III,

When, $T_C \le T_L$ $\quad A_f = KS_{D1}\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0\left(\frac{T_S}{T_C}\right)$

When, $T_C > T_L$ $\quad A_f = KS_{D1}\left(\frac{T_L}{T_C^2}\right) = 2.5KQF_aS_0\left(\frac{T_ST_L}{T_C^2}\right)$

App. E (Freeboard E.7.2)

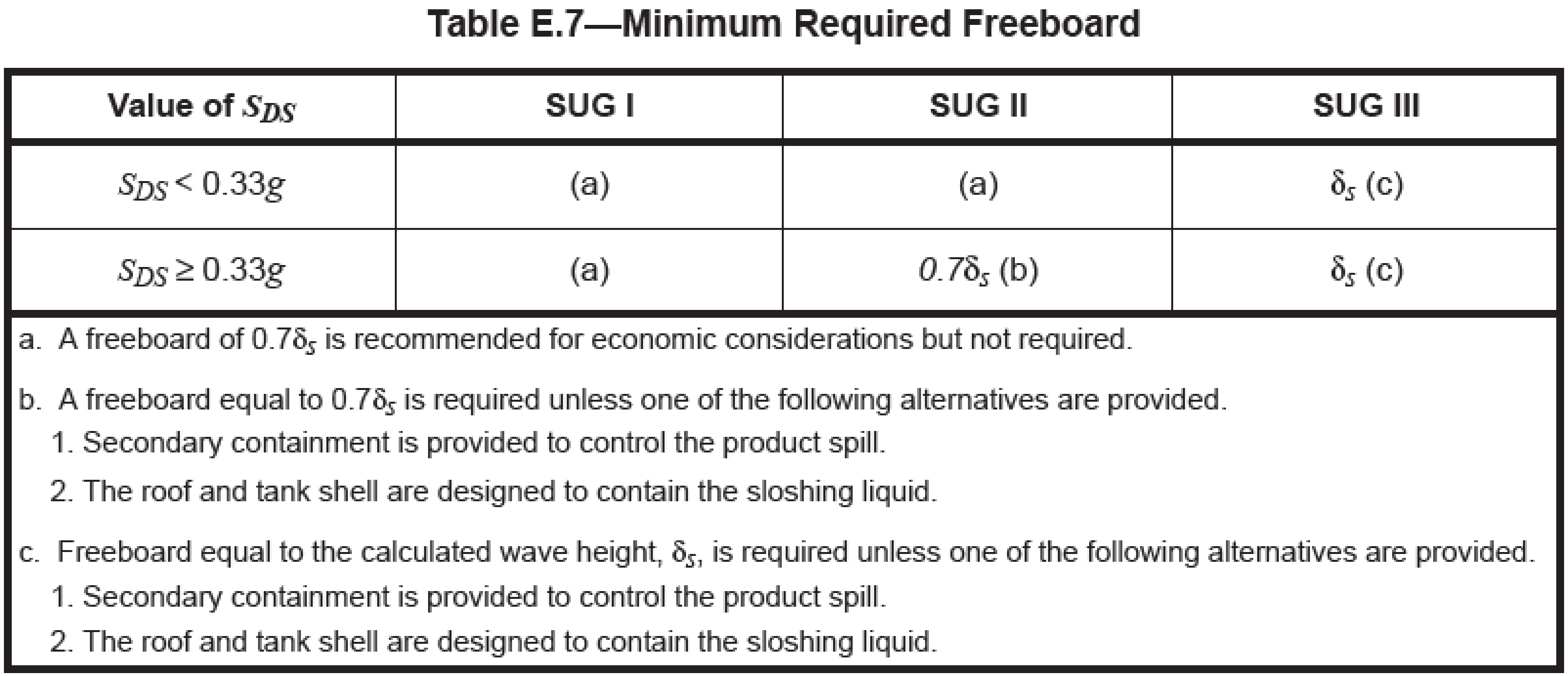


Table E.7—Minimum Required Freeboard

Value of S_{DS}	SUG I	SUG II	SUG III
$S_{DS} < 0.33g$	(a)	(a)	δ_s (c)
$S_{DS} \geq 0.33g$	(a)	$0.7\delta_s$ (b)	δ_s (c)

a. A freeboard of $0.7\delta_s$ is recommended for economic considerations but not required.

b. A freeboard equal to $0.7\delta_s$ is required unless one of the following alternatives are provided.
   1. Secondary containment is provided to control the product spill.
   2. The roof and tank shell are designed to contain the sloshing liquid.

c. Freeboard equal to the calculated wave height, δ_s, is required unless one of the following alternatives are provided.
   1. Secondary containment is provided to control the product spill.
   2. The roof and tank shell are designed to contain the sloshing liquid.

ANCHOR BOLT DESIGN

Anchor Bolt design

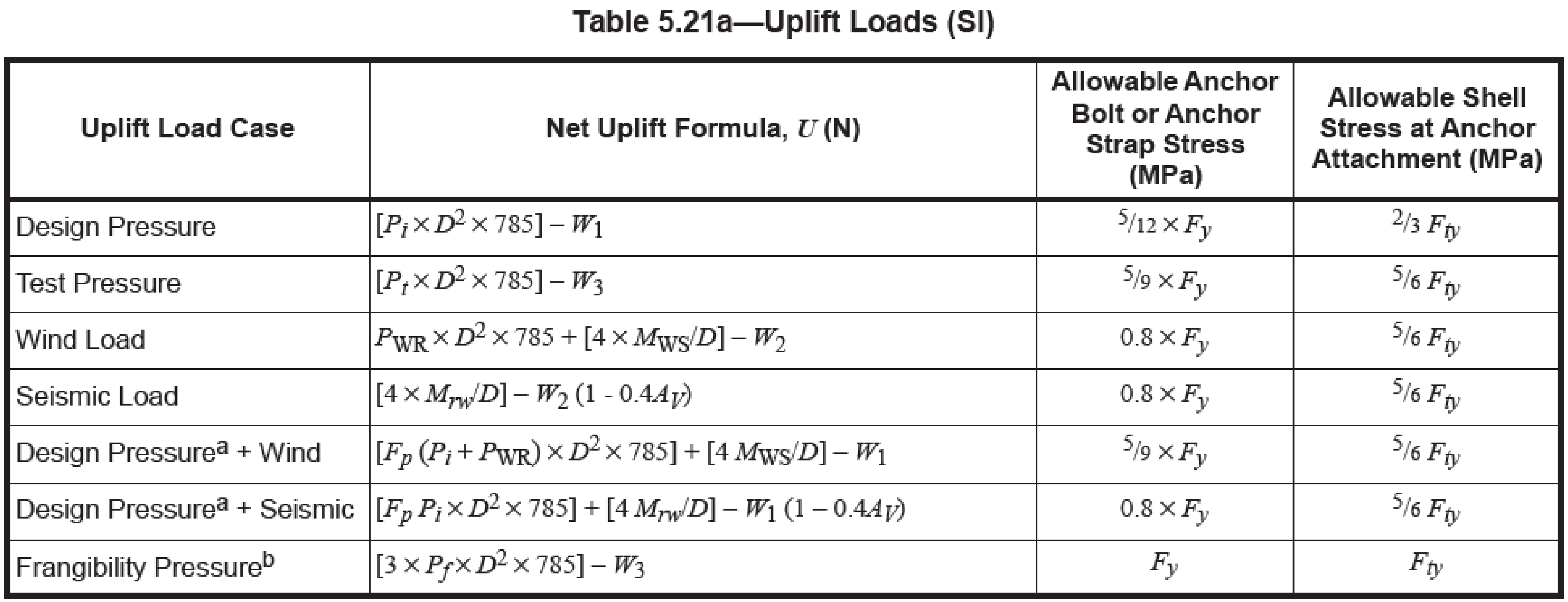


Table 5.21a—Uplift Loads (SI)

Uplift Load Case	Net Uplift Formula, U (N)	Allowable Anchor Bolt or Anchor Strap Stress (MPa)	Allowable Shell Stress at Anchor Attachment (MPa)
Design Pressure	$[P_i \times D^2 \times 785] - W_1$	$^5/_{12} \times F_y$	$^2/_3\ F_{ty}$
Test Pressure	$[P_t \times D^2 \times 785] - W_3$	$^5/_9 \times F_y$	$^5/_6\ F_{ty}$
Wind Load	$P_{WR} \times D^2 \times 785 + [4 \times M_{WS}/D] - W_2$	$0.8 \times F_y$	$^5/_6\ F_{ty}$
Seismic Load	$[4 \times M_{rw}/D] - W_2\ (1 - 0.4A_V)$	$0.8 \times F_y$	$^5/_6\ F_{ty}$
Design Pressure[a] + Wind	$[F_p\ (P_i + P_{WR}) \times D^2 \times 785] + [4\ M_{WS}/D] - W_1$	$^5/_9 \times F_y$	$^5/_6\ F_{ty}$
Design Pressure[a] + Seismic	$[F_p\ P_i \times D^2 \times 785] + [4\ M_{rw}/D] - W_1\ (1 - 0.4A_V)$	$0.8 \times F_y$	$^5/_6\ F_{ty}$
Frangibility Pressure[b]	$[3 \times P_f \times D^2 \times 785] - W_3$	F_y	F_{ty}

- tb = U/N
- tb is the load per anchor;
- S (actual)= tb / bolt area

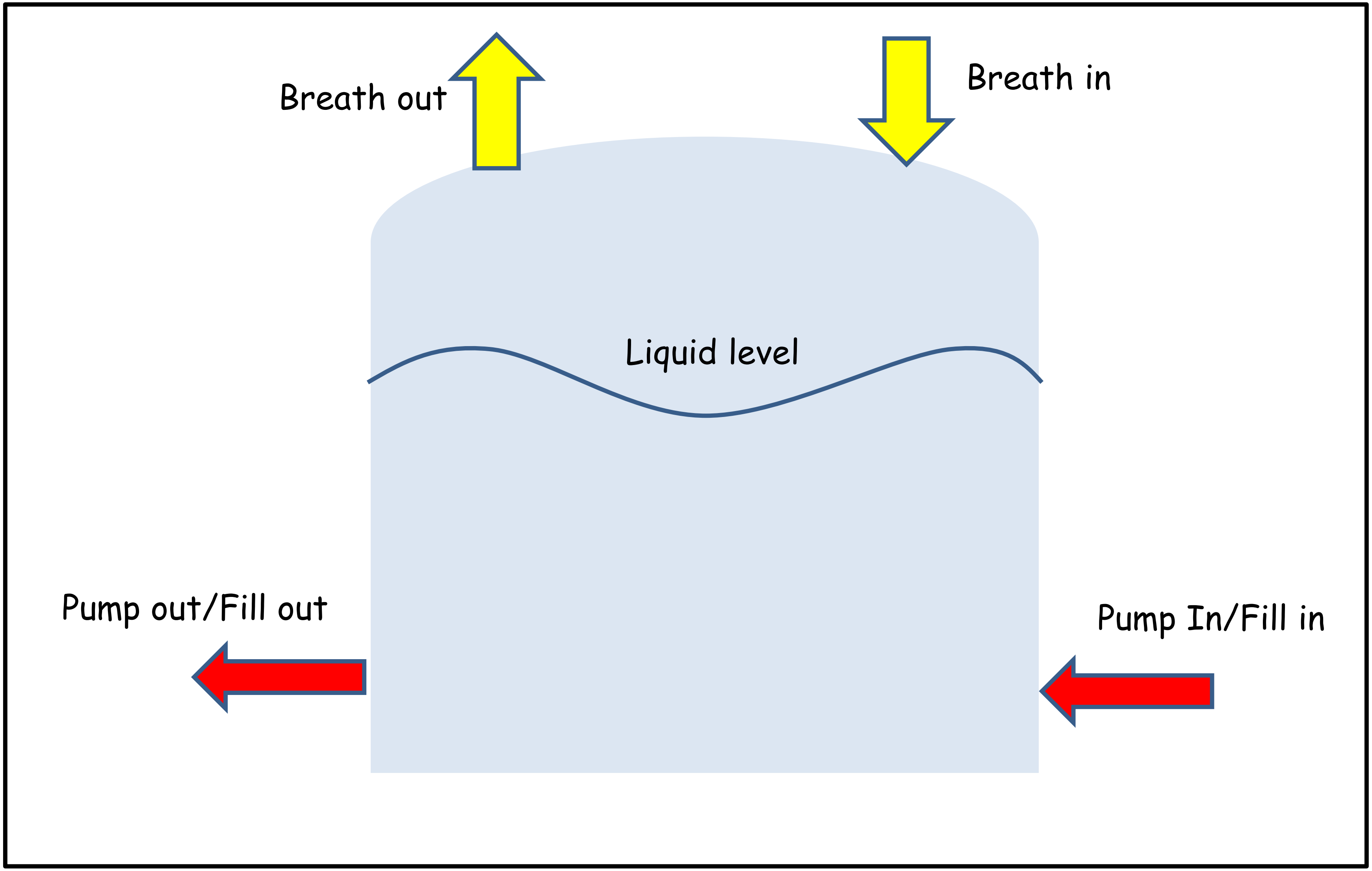
Anchor Bolt design

- **5.12.5** When anchor bolts are used, they shall have a corroded shank diameter of no less than 25 mm (1 in.).
- Carbon steel anchor straps shall have a nominal thickness of not less than 6 mm (1/4 in.) and shall have a minimum corrosion allowance of 1.5 mm (1/16 in.) on each surface for a distance at least 75 mm (3 in.), but not more than 300 mm (12 in.) above the surface of the concrete.

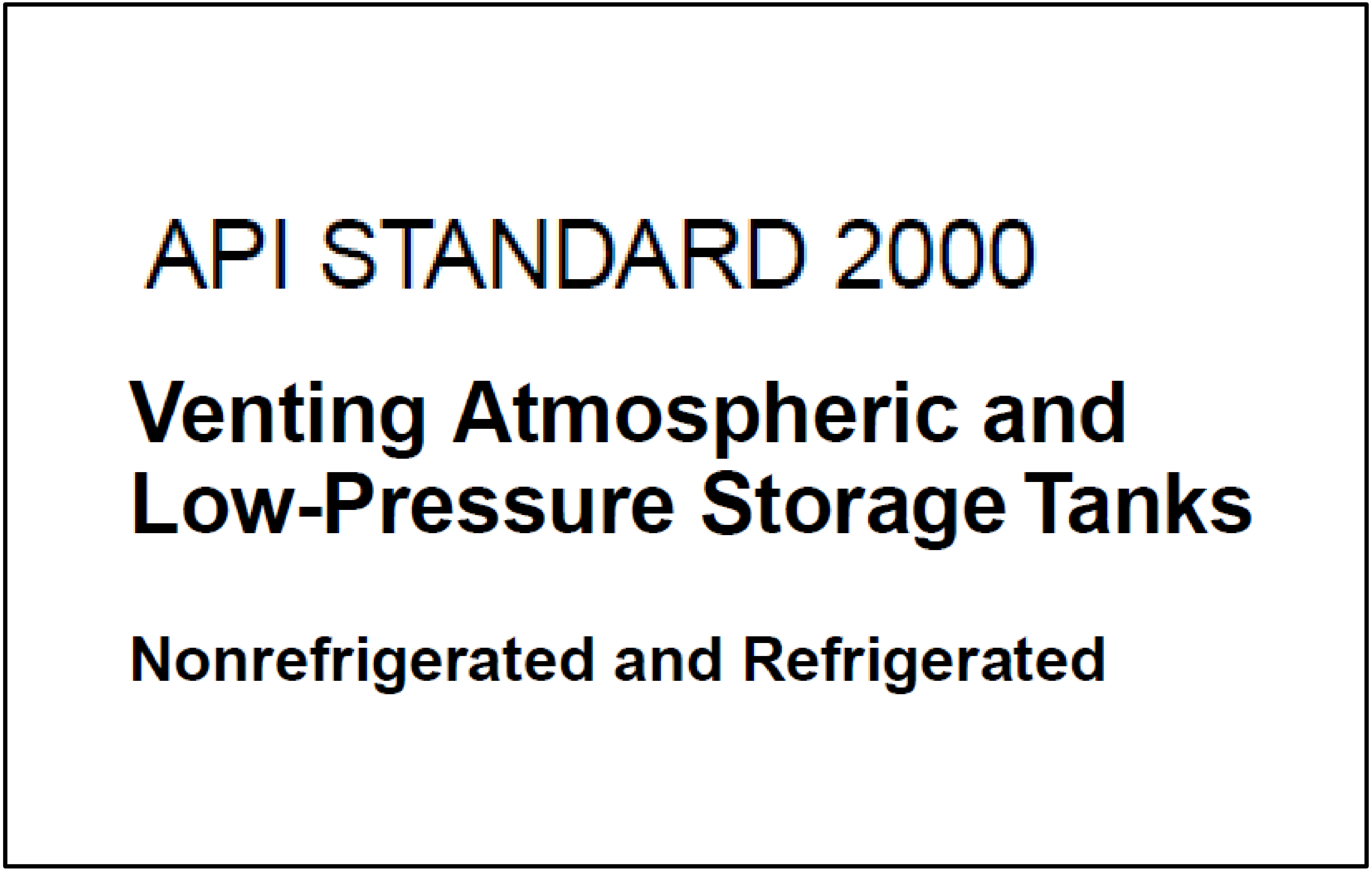
- N is the number of equally spaced anchors. If not equally spaced, then tb shall be increased to account for unequal spacing (a minimum of 4 anchors are required).
- The anchor center-to-center spacing measured along the tank circumference at shell outer diameter shall not exceed 3 m

FREE VENT DESIGN

API 2000 (Vent Design)



API 2000 (Vent Design)



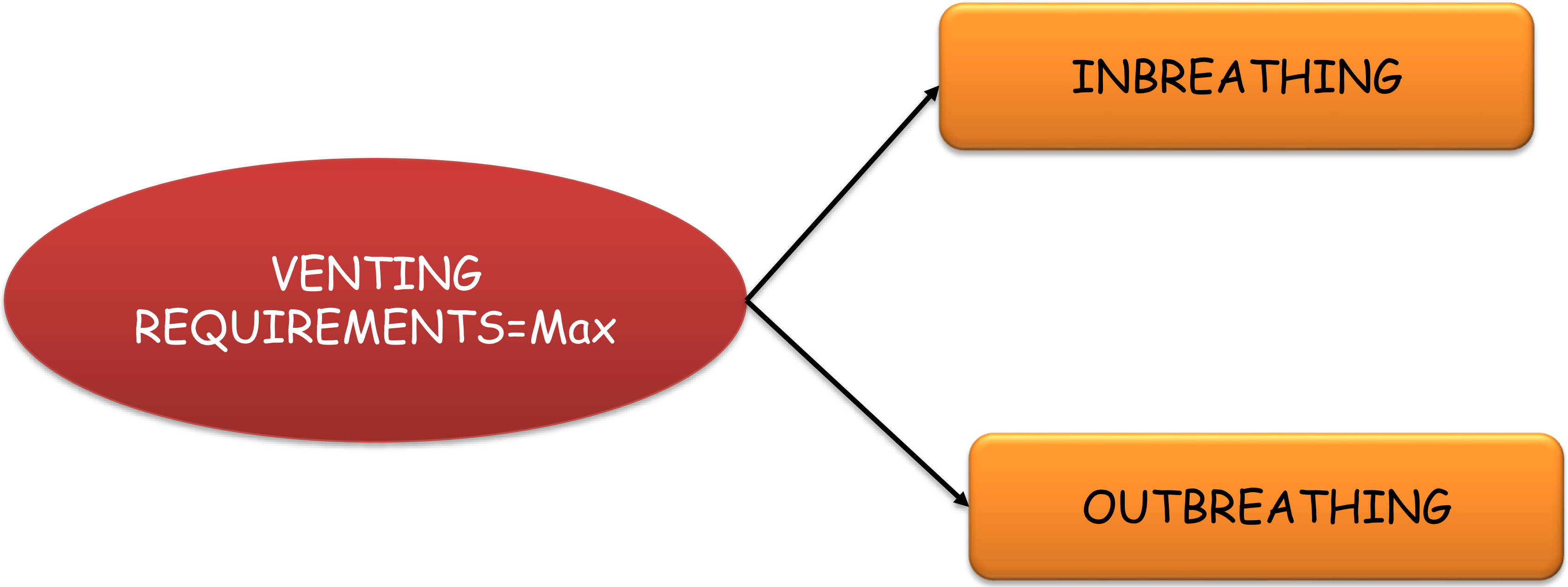
API STANDARD 2000

Venting Atmospheric and Low-Pressure Storage Tanks

Nonrefrigerated and Refrigerated

API 2000 (Vent Design)

Inbreathing (Vacuum Relief)



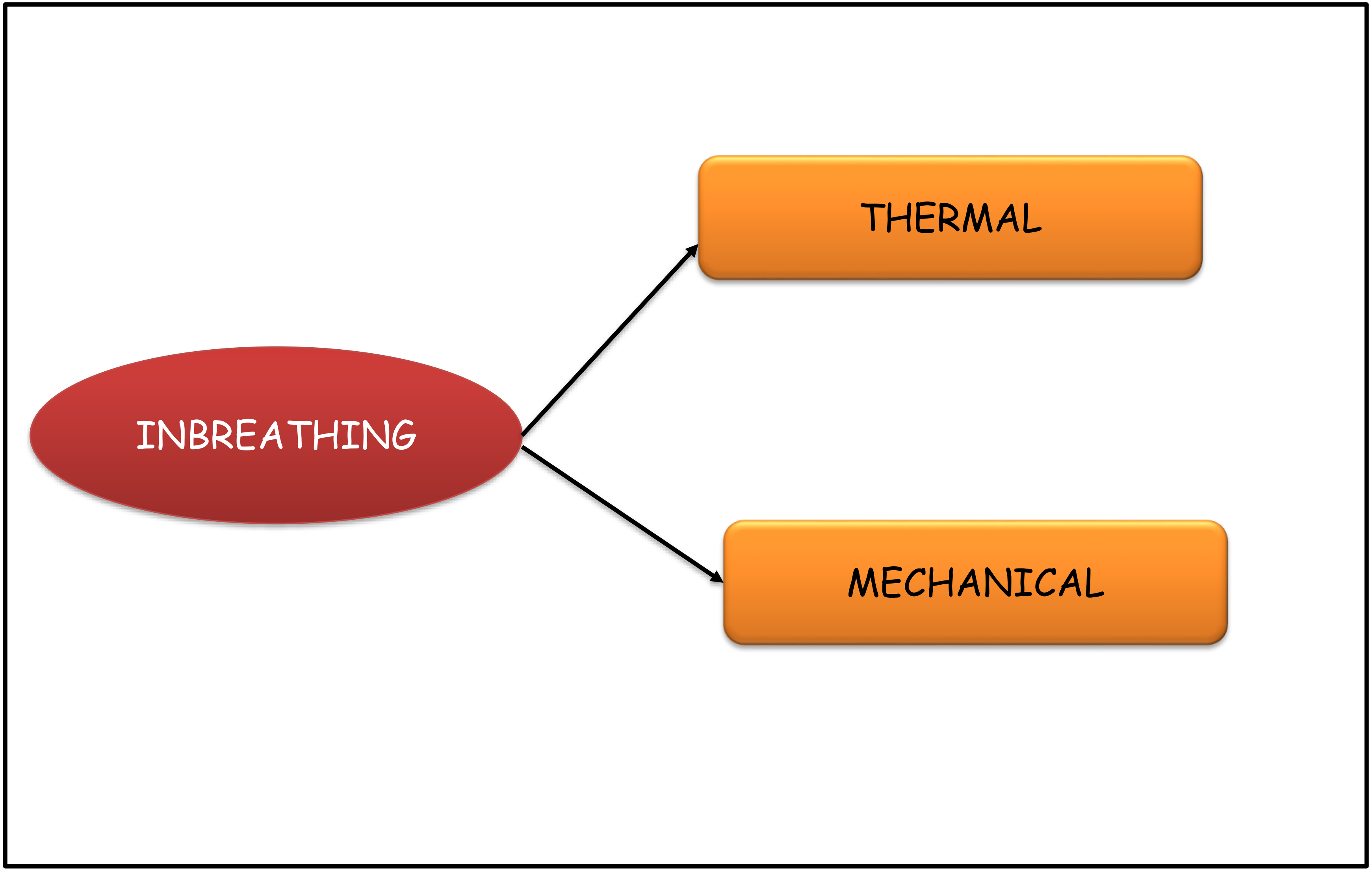
VENTING REQUIREMENTS=Max

INBREATHING

OUTBREATHING

Outbreathing (Pressure Relief)

API 2000 (Vent Design)

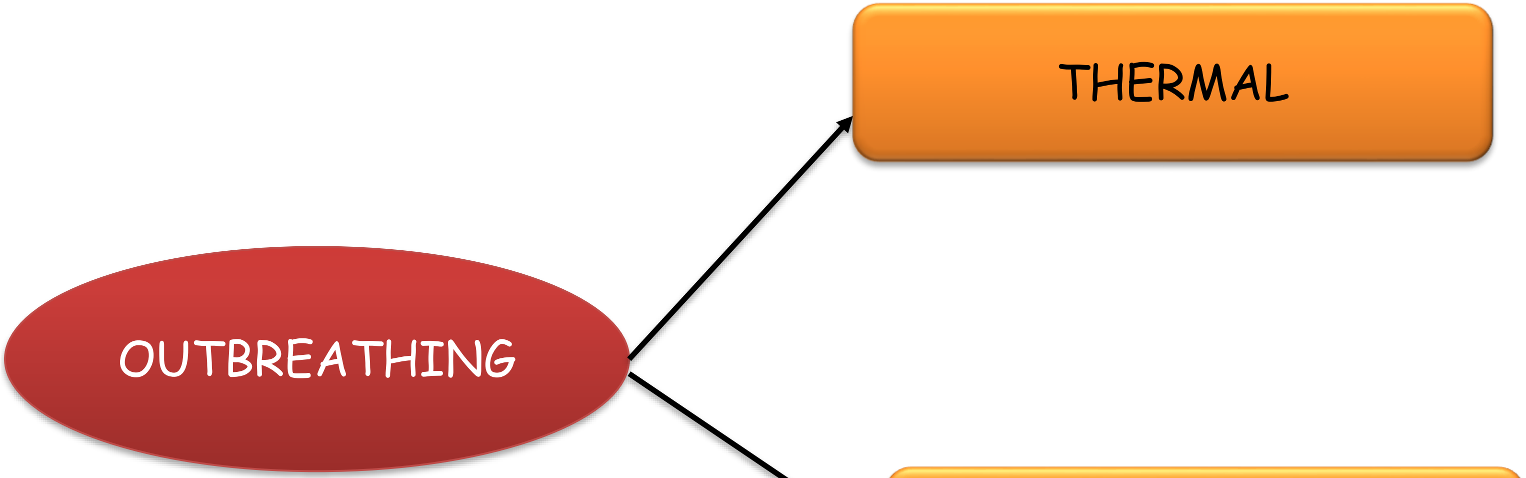


INBREATHING

THERMAL

MECHANICAL

API 2000 (Vent Design)



OUTBREATHING

THERMAL

MECHANICAL

API 2000 (Vent Design)

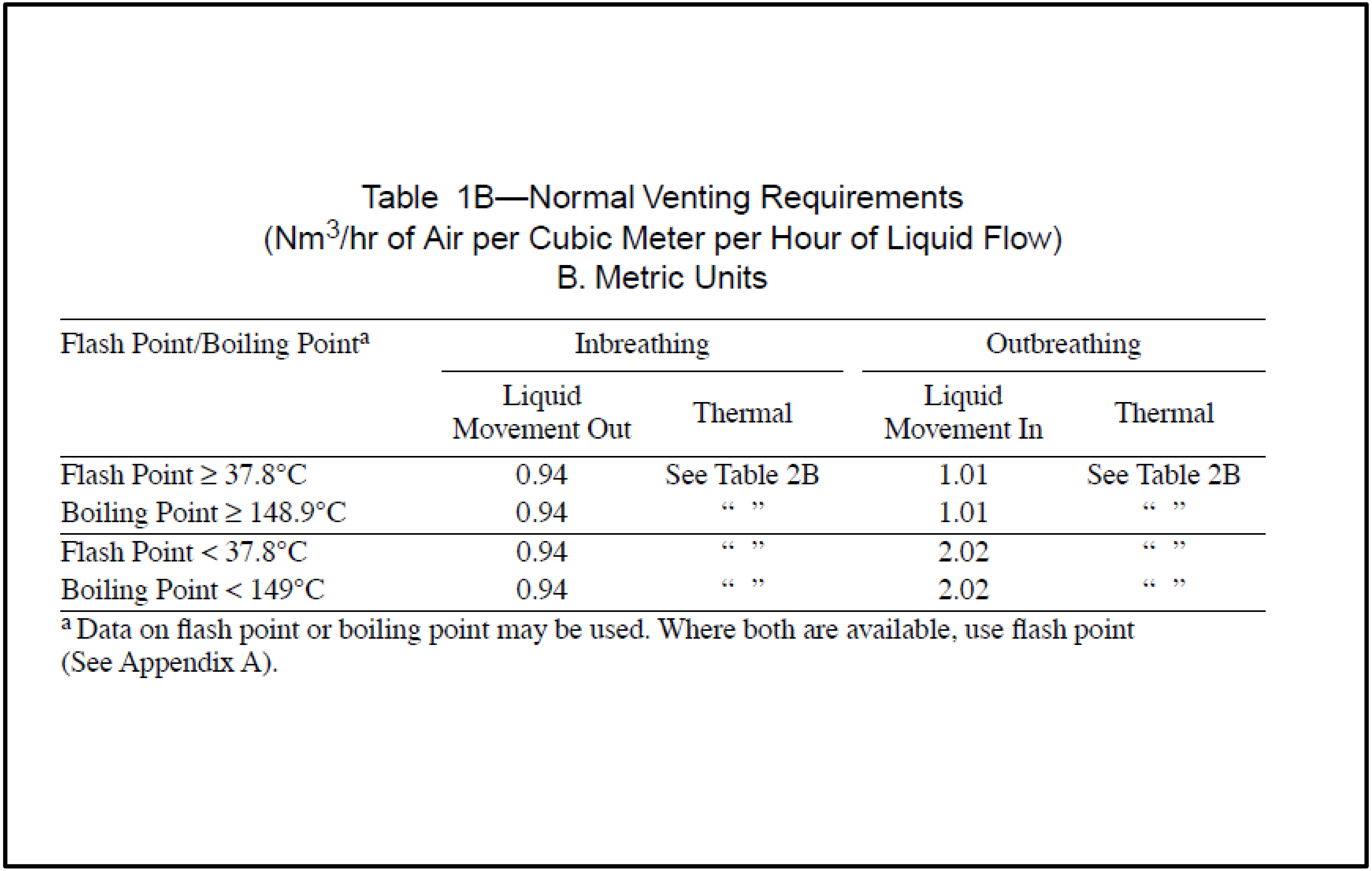


Table 1B—Normal Venting Requirements
(Nm^3/hr of Air per Cubic Meter per Hour of Liquid Flow)
B. Metric Units

Flash Point/Boiling Point[a]	Inbreathing		Outbreathing	
	Liquid Movement Out	Thermal	Liquid Movement In	Thermal
Flash Point ≥ 37.8°C	0.94	See Table 2B	1.01	See Table 2B
Boiling Point ≥ 148.9°C	0.94	" "	1.01	" "
Flash Point < 37.8°C	0.94	" "	2.02	" "
Boiling Point < 149°C	0.94	" "	2.02	" "

[a] Data on flash point or boiling point may be used. Where both are available, use flash point (See Appendix A).

Table 2B —Requirements for Thermal Venting Capacity
B. Metric Units

Tank Capacity	Inbreathing (Vacuum)	Outbreathing	
Column 1[d]	Column 2[a]	Column 3[b]	Column 4[c]
		Flash Point ≥ 37.8°C or Normal Boiling Point ≥ 148.9°C	Flash Point < 37.8°C or Normal Boiling Point < 148.9°C
Cubic Meters	Nm^3/h	Nm^3/h	Nm^3/h
10	1.69	1.01	1.69
20	3.37	2.02	3.37
100	16.9	10.1	16.9
200	33.7	20.2	33.7
300	50.6	30.3	50.6
500	84.3	50.6	84.3
700	118	70.8	118
1,000	169	101	169
1,500	253	152	253
2,000	337	202	337
3,000	506	303	506
3,180	536	388	536
4,000	647	472	647
5,000	787	537	787
6,000	896	602	896
7,000	1,003	646	1,003
8,000	1,077	682	1,077
9,000	1,136	726	1,136
10,000	1,210	807	1,210
12,000	1,345	888	1,345
14,000	1,480	969	1,480
16,000	1,615	1,047	1,615
18,000	1,745	1,126	1,745
20,000	1,877	1,307	1,877
25,000	2,179	1,378	2,179
30,000	2,495	1,497	2,495

API 2000 (Vent Design)

$Q = VA$

V = Air velocity (5 ~ 15 m/s)

A = Cross section area of Nozzle