British Plastics Federation
PIPES GROUP

Guide to Designing Geocellular Drainage Systems to CIRIA Report C737

September 2018

Contents

It has been assumed in the drafting of this guidance that the execution of its provisions is entrusted to appropriately qualified and experienced people. Compliance with this guide does not itself confer immunity from legal obligations and all relevant National Legislation and Standards apply.

Information contained in this guidance is given in good faith. The British Plastics Federation (BPF) Pipes Group cannot accept any responsibility for actions taken by others as a result.

1. Introduction

CIRIA Report C737, *Structural and Geotechnical Design of Modular Geocellular Drainage Systems*, was published in 2016 and is a key reference of *The SuDS Manual* (CIRIA Report C753, 2015). Prior to publication of C737, the design of many geocellular drainage systems followed the guidance in CIRIA Report C680, *Structural Design of Modular Geocellular Drainage Tanks* (CIRIA, 2008). The C680 approach has been in use since around 2001 and the performance of the tanks designed to this method over the past 17 years has shown it to be a pragmatic and robust approach to the design of geocellular tanks. At the time of publication of this guide, the British Board of Agrément (BBA) certificates for geocellular units were based on the principles described in C680. In time, it is anticipated that once appropriate standards are in place for testing, BBA would move towards the design approach in C737.

This BPF Pipes Group guide is intended to aid the designer of geocellular drainage systems in the application of C737 using a case study and a worked example.

The main differences in approach between the worked example in this guide, in C737 and in C680 are summarised in Appendix A of this guide.

Throughout this guide, the key sections of C737 to be used are highlighted. This guide must be read in conjunction with both C737 and *The SuDS Manual*. *The SuDS Manual* can be downloaded free of charge from the website www.susdrain.org.

Note: The hydraulic design and sizing of the tank are outside the scope of this guide. The hydraulic sizing methods described in *The SuDS Manual*, local design guides or standards should be used.

2. Design process

2.1 Process

(Figure 21.17 *The SuDS Manual*)

This guide follows the process on the adjacent page which is based on **Figure 21.17 of *The SuDS manual* (2015)**.

Preliminaries

Before the design commences it is necessary, as the first stage of the process, to appoint a designer under contract. The appointment to provide design services under contract is important to ensure there is a clear understanding of who is responsible for the design of the tank.

The Construction (Design and Management) Regulations apply to all construction projects. The process in this example is consistent with the requirements of the CDM Regulations 2015. For notifiable projects under the CDM Regulations 2015 (i.e., work that is expected to last more than 30 days and have more than 20 workers working at the same time at any point on the project or exceed 500 person days of construction work) additional duties apply.

The Client should appoint a Principal Designer. The Client should provide all the relevant information to the Principal Designer. The Principal Designer should either carry out the design of geocellular tanks or make sure that another suitably-qualified organisation is appointed. The designer of a tank may be a consultant, contractor or supplier. The important thing to note is that unless there is a contract to complete the design work, the designer may not be liable for any problems later. Some suppliers offer a design, supply and install package and in this case the contract documents should clearly specify the design responsibility.

C737 Process Steps

Step 1 – Determine the qualifications of the designer along with the testing, analysis and design checks that are required dependant on the site classification (0 to 3).

Step 2 – Prepare a conceptual ground model which summarises the critical factors relevant to the design (geology, soil and tank parameters, tank geometry, etc.). This should be a diagrammatic cross-section.

Step 3 – Determine the loads that are **realistically** likely to be applied to the tank. A conservative approach is applied throughout C737 and engineering judgement may determine that some assumptions are not applicable to a site (e.g., the assumption that a tank in a garden next to a drive will be subject to HGV loads). Apply appropriate partial factors of safety to obtain the design loads.

Step 4 – Determine the characteristic strength and deformation properties for the geocellular units. Manufacturers should provide sufficient information to allow designers to understand and analyse the performance of the units. The parameters should be those that are declared by the manufacturer. Apply appropriate partial factors of safety to obtain design properties.

Step 5 – Compare the design loads to the design strength. Assess elastic deformation under short-term loads and permanent deformation under long-term loads.

Step 6 – Prepare a geotechnical design report. This does not have to be a long-winded report. The purpose of the report is to communicate to those building the tank the critical aspects of the design approach and assumptions made that they need to be aware of. The most effective form of communication is a short one- or two-page summary of the information (including a diagrammatic ground conceptual model).

C737 Process	C737 Pages/Figures/Tables	C737 Forms
Preliminaries Appoint designer under contract Provide relevant design information	50, 70, 149, 150	Project Roles and Sign Off Sheet Designer Evaluation Form
STEP 1 Determine site classification Determine design class and design/checking requirements	43 – 50, 71, 146, 147, 148 Tables 3.2, 3.3 and 5.1	Design and Construction Classification and Check Proforma
STEP 2 Develop the conceptual ground model	75, 78 - 82 Figure 5.2	None
STEP 3 Determine/calculate characteristic applied loads for transient, permanent and accidental conditions (vertical and horizontal) Apply factors of safety to applied loads for ultimate and serviceability limit states	37 – 40, 82 – 96, 98 - 100	None
STEP 4 Determine characteristic strength and deformation properties for the units from test data Apply factors of safety to the properties for ultimate and serviceability limit states	58 – 64, 64 - 67, 76 – 78, 151	Product Evaluation Form
STEP 5 Design calculations and analysis Compare design strength to design loads and deformation to acceptable limits	102 – 106	None
STEP 5a Global deformation and site stability assessment	100, 164 - 166	None
STEP 6 Prepare geotechnical design report	114, 115	None

2.2 Accidental loading

C737 requires the designer to consider routine loads (i.e., the standard load case) and the performance of the tank under accidental loads. The accidental load analysis uses higher loads but lower factors of safety than the standard load case. Examples of an accidental load are an HGV entering a car park that is only designed for car traffic or materials being temporarily stockpiled on a tank during construction when the tank should be fenced off to prevent this.

In this worked example, calculations are shown that analyse a standard load case. The same process should also be repeated for the accidental load scenario using the accidental loads and appropriate partial factors of safety.

2.3 Temporary construction situation

In this worked example, it is assumed that the tank would not be subject to traffic during construction until the final car park surfacing has been laid. It is also assumed it will not be trafficked by cranes or cherry pickers. If the tank will be trafficked by construction traffic when the cover is less than the final design and/or by heavier vehicles than those expected in service, a separate set of calculations should be completed using appropriate loads and factors of safety.

3. Details for the worked example in this guide

The worked example in this guide is based on the information provided below.

Example site – BPF Towers

A tank is to be installed below a car park for a supermarket, at a depth of 2.4 m to the invert level of the tank (or base of tank). There are no height barriers in the car park but warning signs will be provided prohibiting HGVs from the car park area where the tank is situated. The cover over the top of the tank to the top of the car park surfacing (finished ground level) is 1.2 m which is consistent over the whole tank. The tank will be 30 m long by 10 m wide by 1.2 m high.

The tank will be an attenuation tank installed in level ground. The nearest building to the tank is 5.5 m away and the toe of a railway embankment is located 15 m from the tank. The tank will be wrapped in a geomembrane (i.e., a waterproof liner). The site and tank layout is shown in Figure 1.

The scheme drawings showing the site layout, drainage layout, sections and details have been provided to the Principal Designer along with the ground investigation report, which includes information on the groundwater conditions.

The ground conditions at the tank site comprise:

- Made Ground – typically 1m thick and comprising medium dense black sandy GRAVEL of ash and clinker.
- Glacial Till – typically 6 m thick and comprising firm to stiff dark grey silty CLAY with much fine to coarse gravel.
- Coal Measures – not investigated but typically comprises a series of sandstones, siltstones, mudstones and coal seams. Features that could affect tank stability such as shallow coal workings or shafts are not expected.

Groundwater monitoring has shown that groundwater is not anticipated to be present above the base of the tank at any point during the year.

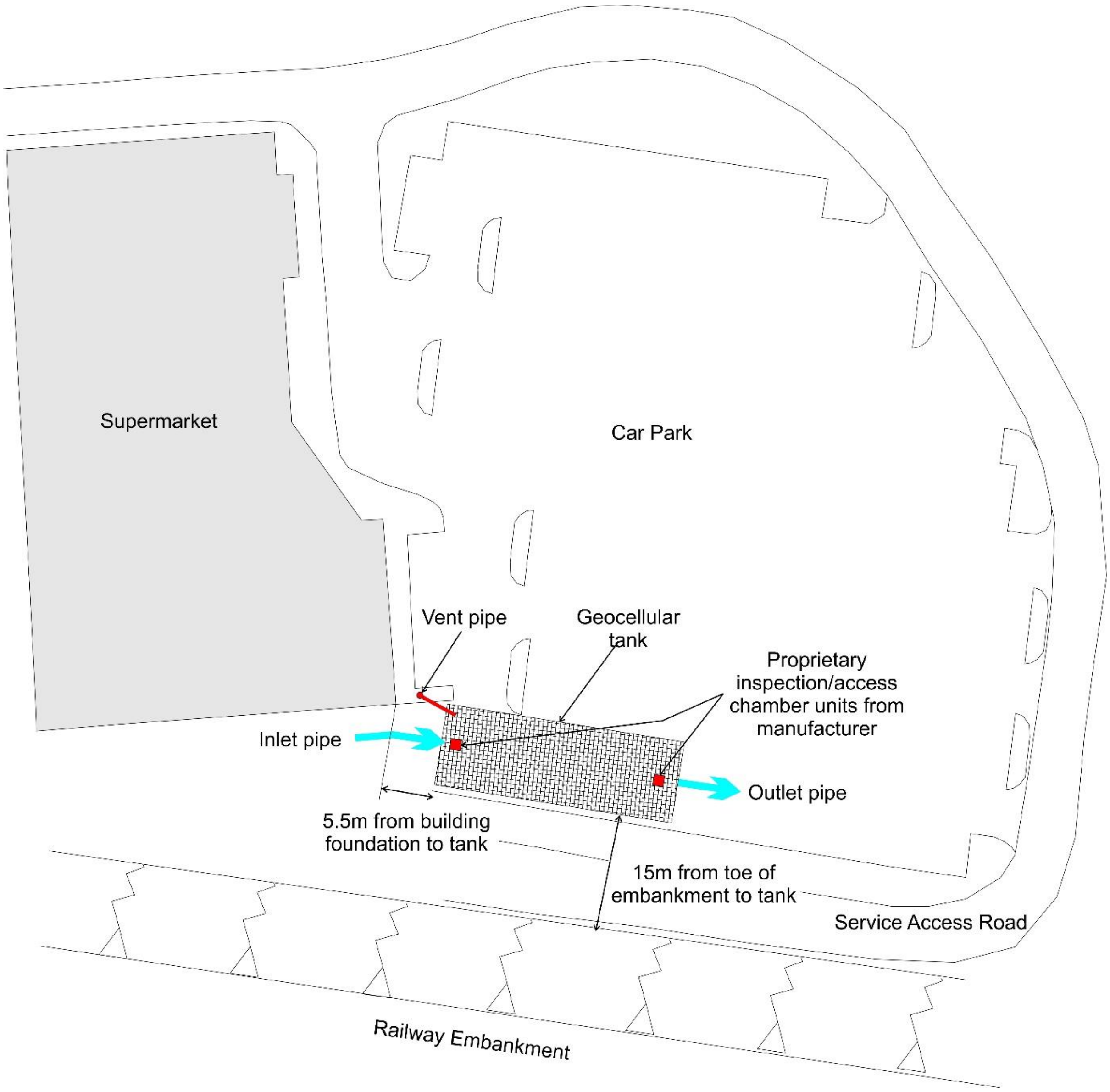
The tank will be installed in an excavation that has a 0.5 m wide working space at the bottom and with slopes battered back at 1 in 1. The excavation around the sides of the tank will be backfilled with Class 6N Material (Manual of Contract Documents for Highway Works, Volume 1, Specification for Highway Works). It is intended that once the tank is backfilled and constructed to pavement level that construction traffic will pass over it but it is not in a location where cranes, etc., are likely to operate. The road/car park pavement construction will comprise 100 mm of asphalt over 200 mm of Type 1 sub-base. The remaining depth of fill to the top of the tank will comprise general granular fill material.

The tank will be provided with a vent consisting of a 100 mm pipe in a suitable location that is accessible.

Inlet and outlet details and maintenance access are shown on the scheme general arrangement drawing.

The geocellular units to be used in this example are manufactured by Mr Plastic Manufacturing Company Limited. WaterBox 1 Units will be supplied. A 50-year design life has been specified for the tank by the Client.

Figure 1 Example scheme general arrangement

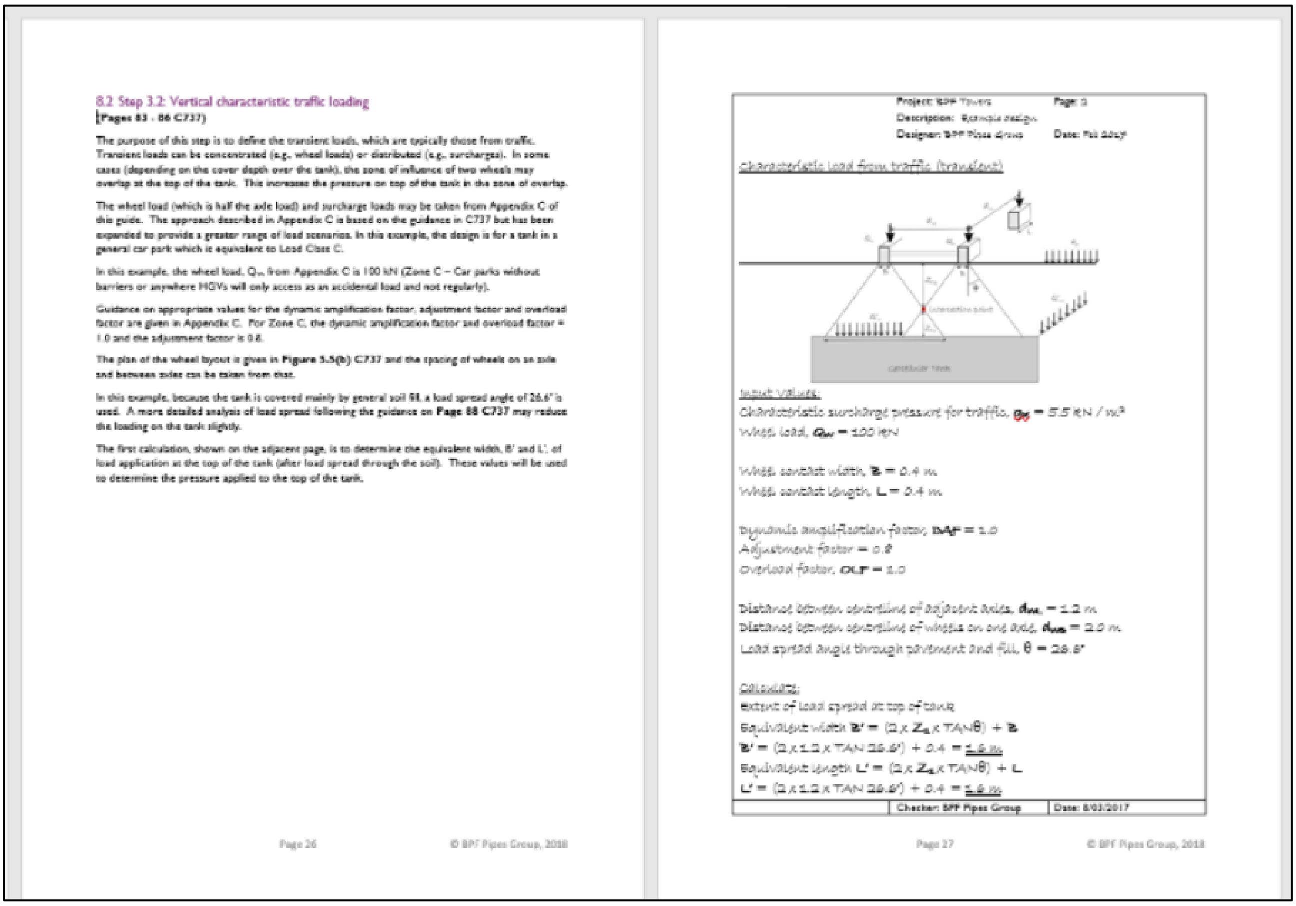


4. Layout of the worked example in this guide

The layout of this guide in the following pages is shown below.

Explanation of the forms or calculations with references to the relevant pages in C737 and *The SuDS Manual*

Example of the completed form or calculation



8.2 Step 3.2: Vertical characteristic traffic loading

(Pages 83 - 86 C737)

The purpose of this step is to define the transient loads, which are typically those from traffic. Transient loads can be concentrated (e.g., wheel loads) or distributed (e.g., surcharges). In some cases (depending on the cover depth over the tank), the zone of influence of two wheels may overlap at the top of the tank. This increases the pressure on top of the tank in the zone of overlap.

The wheel load (which is half the axle load) and surcharge loads may be taken from Appendix C of this guide. The approach described in Appendix C is based on the guidance in C737 but has been expanded to provide a greater range of load scenarios. In this example, the design is for a tank in a general car park which is equivalent to Load Class C.

In this example, the wheel load, Q_w, from Appendix C is 100 kN (Zone C – Car parks without barriers or anywhere HGVs will only access as an accidental load and not regularly).

Guidance on appropriate values for the dynamic amplification factor, adjustment factor and overload factor are given in Appendix C. For Zone C, the dynamic amplification factor and overload factor = 1.0 and the adjustment factor is 0.8.

The plan of the wheel layout is given in **Figure 5.5(b) C737** and the spacing of wheels on an axle and between axles can be taken from that.

In this example, because the tank is covered mainly by general soil fill, a load spread angle of 26.6° is used. A more detailed analysis of load spread following the guidance on **Page 88 C737** may reduce the loading on the tank slightly.

The first calculation, shown on the adjacent page, is to determine the equivalent width, B' and L', of load application at the top of the tank (after load spread through the soil). These values will be used to determine the pressure applied to the top of the tank.

Page 26 © BPF Pipes Group, 2018

Project: BPF Towers Page: 1
Description: Example design
Designer: BPF Pipes Group Date: Feb 2017

Characteristic load from traffic (transient)

Input values:
Characteristic surcharge pressure for traffic, q_{tr} = 5.5 kN / m²
Wheel load, Q_w = 100 kN

Wheel contact width, B = 0.4 m
Wheel contact length, L = 0.4 m

Dynamic amplification factor, DAF = 1.0
Adjustment factor = 0.8
Overload factor, OLF = 1.0

Distance between centreline of adjacent axles, d_{wL} = 1.2 m
Distance between centreline of wheels on one axle, d_{wB} = 2.0 m
Load spread angle through pavement and fill, θ = 26.6°

Calculate:
Extent of load spread at top of tank
Equivalent width B' = (2 x Z_s x TANθ) + B
B' = (2 x 1.2 x TAN 26.6°) + 0.4 = 1.6 m
Equivalent length L' = (2 x Z_s x TANθ) + L
L' = (2 x 1.2 x TAN 26.6°) + 0.4 = 1.6 m

Checker: BPF Pipes Group Date: 8/03/2017

Page 27 © BPF Pipes Group, 2018

References to relevant pages or tables in **C737** or ***The SuDS Manual*** are shown in bold

5. Preliminaries

Prior to starting the design, the *Project Roles and Sign Off Sheet* and the *Designer Evaluation Form* should be completed (as far as is possible at this stage).

5.1 Project Roles and Sign Off Sheet

(Pages 50, 70, 149 C737)

The *Project Roles and Sign Off Sheet* identifies the main parties in the design and installation of a geocellular tank. It will be a living document and should be first used to record the details of the designer of the tank. As the project progresses, the other parties can be added as they become known. A copy of the sheet from **Appendix A1 C737** is provided on the adjacent page.

The **Client** is the person who is commissioning the design and construction of the project.

The **Principal Designer** is the organisation that is responsible for the structural and geotechnical design of the tank. This may be the consultant that has designed the overall drainage system or it may be delegated to a specialist sub-consultant or supplier/manufacturer. In this example, it is Drainage Design Consultant Limited.

The **Principal Contractor** is that organisation designated under the CDM Regulations. In this example the design is being completed before tendering and, therefore, the Principal Contractor is not yet known.

The **Geocellular Manufacturer/Supplier** is the organisation that supplies the tank units. If this changes during the development of the project (for example, if the Principal Contractor proposes an alternative system to that shown in the design or a minimum performance specification has been provided by the designer) then this form should be updated. In this example, it is Mr Plastic Manufacturing Company Limited.

Site classification assessment is based on the results of the *Design and Construction Classification and Check Proforma* (see the next section of this guide). In this example, the results of completing the *Design and Classification and Check Proforma* indicate the site is Class 1.

SITE CLASSIFICATION PROFORMA

Project roles and sign off sheet

Project title BPF TOWERS

Site location address BPF TOWERS, No 10 DOWNING ST, LONDON

Client

Name and organisation BIG DEVELOPER LTD

Office 020 7457 5000 Mobile 07779 100100

Email plastic@bpf.co.uk.

Principal designer

Name and organisation DRAINAGE DESIGN CONSULTANTS LTD

Office 020797458 Mobile 07779 100101

Email DDCL@bpf.co.uk

Principal contractor

Name and organisation NOT KNOWN AT THIS TIME

Office — Mobile —

Email —

Geocellular manufacturer/supplier

Name and organisation MR PLASTICS MANUFACTURING COMPANY LTD

Office 01709 770000 Mobile 07779 100102

Email info@mrplastic.com

Site classification assessment

Class 0 ☐ **Class 1** ☐ X **Class 2** ☒ **Class 3** ☐

	Name	Signature	Date
Client	MR BOSS	The Boss	17/1/2017
Principal designer	MR MATHS	[illegible]	18/1/2017
Principal contractor	—	—	—
Manufacturer	MR BOX	[illegible]	19/1/2017.

Note:
1. Where there is a domestic client, the principal contractor will usually undertake the client role. A domestic client can also give written authority to a designer or manufacturer, in which case they can become the principal designer.

5.2 Designer Evaluation Form

(Page 150 C737)

This form is used to summarise the relevant design information that has been passed to the Principal Designer by the Client or other party (e.g., main design consultant).

The design information for the worked example is summarised in the form on the adjacent page.

Design function – in this example, the tank is an attenuation tank.

End surface use – in this example, the tank will be below a supermarket car park which can be defined as a ‘car park, general, no height access restrictions’. Judgement should be applied into which category a site fits. Careful consideration of likely access by HGVs is required, as factors other than the height of barriers may restrict access (e.g., very tight corners, width of access route, earth berms or planting around landscaped areas, etc.).

Background information provided to the manufacturer – in this example, it is assumed that all necessary information has been provided. If information is missing then any assumptions made in the design or caveats as to its application should be clearly stated. In this case, the dimensions for the tank are shown as 30 m x 10 m x 1.2 m. The ground is level and so the maximum and minimum depth of cover is the same at 1.2 m and the finished ground level (FGL) variation is zero.

Volume of installation – this is termed ‘net volume’ in the **C737 *Design Evaluation Form***. The usual understanding of the term ‘net volume’ would be the storage volume required, with ‘gross volume’ being the total volume of the tank considering porosity. In the form there is no space to include a value for porosity, therefore, the volume of installation is simply the volume of the tank. This has no practical significance to the design.

Construction details provided to the manufacturer – it is important that any construction details assumed or required in the design are stated. For example, in this case the assumption of the use of Class 6N backfill will affect the angle of friction and hence the applied lateral pressure on the side of the tank. These factors should also be carried forward to the geotechnical design report. Details of maintenance access points to inspect or clean the tank, inlets and outlets and ventilation of the tank are shown on the scheme general arrangement drawings.

SITE CLASSIFICATION PROFORMA

Designer evaluation form

Proforma checklist information to be passed on to geocellular manufacturer and principal designer, prepared by the installation designer/ architect/specifier. **NB: pre design/construction information held, to be passed to geocellular manufacturer.**

Project title BPF TOWERS

Design function

Attenuation [X] Soakaway [ ] Grey/rainwater storage [ ] Other [ ]

Specify ....

Intended design philosophy	ON LINE ATTENUATION TANK	eg on/offline, sub-base replacement, soakaway, storage, gas venting

End surface use

Residential garden [ ]	Pasture/woodland/ parkland/landscaping [ ]	Arable farmland (tractor/ harvester access) [ ]
Residential driveway [ ]	Car park (light use with height access restrictions) [ ]	Car park general (no height access restrictions) [X]
HGV parks/ low speed roads [ ]	Full highway loading [ ]	Adjacent to existing/planned structures /roads [ ]

Background information provided to manufacturer

Ground investigation [X] Scheme drawings [X] Ground water information [X]

Volume of installation 360 m³ Net* L 30 m W 10 m D 1·2 m

Net volume/voids

Depth of cover Max 1·2 m Min 1·2 m FGL varies in level 0 eg sloping FGL or flat surfacing.

Construction details provided to manufacturer

Predominant geology/soil type	MADE GROUND GLACIAL TILL	eg sand, clay, gravel, peat	**Proposed backfill**	CLASS 6N	eg engineered fill/HA type 1, Class 6N, selected as dug site material.
Foundation proposed	GLACIAL TILL PROOF ROLLED	eg natural ground, sand, lean mix, type 1. Proposed CBR	**TW access/plant tracking on completed or partially complete installation**	NO CRANES GENERAL CONSTᴺ TRAFFIC	Excluding plant used for construction of units
Installation wrapping type	GEOMEMBRANE WELDED	eg welded membrane, filter geotextile, fleece, hybrid system	**Venting requirements**	SEE DWG	eg 1 x 150 mm Dia / 150m² or 1 vent per installation
Maintenance access	YES - SEE DWG		**Inlet / outlet details**	SEE DWG.	eg number and diameter

Further comments/information

150 CIRIA, C737

6. Step 1: Determine site classification, design class and design/checking requirements

(Pages 43 - 50, 146, 147 C737)

6.1 Worked example

The purpose of the *Site Classification Proforma* is to distinguish the level of design and checking that is required. This can range from simple sites that need very little design input to complex sites or sites where the consequences of failure are severe where a high degree of analysis and checking may be necessary.

Experience shows that sudden catastrophic collapse of geocellular structures is not likely to occur and if collapse does occur it would be a slow progressive mechanism. This should be considered when assessing the consequences of failure.

The *Site Classification Proforma* is completed and the site and installation together will achieve a score.

The score is used to define the classification of the site and tank (**Table 3.2, Page 48 C737**).

The classification of the site and tank determines the level of design checking that is necessary (**Table 3.3, Page 49 C737**).

In this example, the site is not within any zones of influence from slopes, retaining walls or foundations. The tank is 5.5 m from the nearest building foundation and the depth, h, is 2.4 m. The limit for the zone of influence is shown on the proforma as 2 m + h = 4.4 m. Therefore, the tank is not within the zone of influence of the foundation.

The tank is 15 m from a railway embankment. The limit for the zone of influence is shown on the proforma as 10 m + h = 12.4 m. Therefore, the tank is not within the zone of influence of the embankment.

1. Type of site - The site in this example is a supermarket and, therefore, is a commercial application. Score = 10.

The single domestic dwelling only applies to small soakaway or attenuation tanks for a single private house.

2. Use - The tank will be an attenuation tank. Score = 5.

The BPF Pipes Group considers that the use of the tank as attenuation or soakaway makes no difference to the level of risk in the structural design. For tanks above the groundwater table, the risks and consequences associated with structural failure are the same for both an attenuation tank or a soakaway and a score of 5 can be used. However, if attenuation tanks are constructed below the water table the risk of failure is higher and so a higher score of 10 is applied. It is preferable to construct all tanks above the water table, wherever possible.

Assign a score based on the level of risk or consequences of failure with respect to the structural design. Attenuation and grey/rainwater storage are given a score of 5 in the proforma rather than 10. For other applications, the score does not have to be 15 as stated on the proforma.

Design and construction classification and check proforma

Proforma objectives and general notes

1. The **PRINCIPLE AIM** of the scoring system is to identify projects with **high intrinsic complexity** and/or where the **consequences of failure** are severe.
2. **CIRIA C737 Chapter 3 Site classification methodology; should be studied before using the proforma.**
3. Proforma provides the methodology to classify installations and recommend the appropriate level of expertise to oversee design and construction.
4. Proforma is user friendly to non-technical clients, highlighting basic checks/risks and guides building professionals to basic design and construction considerations.
5. The methodology utilises the existing Construction Design Management Regulations 2015 (CDM2015), which provides the legal framework and duties for construction projects.
6. Can be used as auditable evidence that the design process, checks and duty of care obligations discharged.
7. The principal designer (CDM2015) will be tasked with the responsibility of ensuring the project proforma are utilised and signed off by the building professional(s) and countersigned by the principal designer. The client or principal designer may delegate this task to the installation designer to manage and ensure completion of the forms.
8. Methodology considers structural and geotechnical design and construction and <u>**NOT**</u> hydraulic design or performance or Environment Agency (EA) consents.
9. This Proforma pack is to be **retained together** so that all appointees are able to review the whole. The enclosed forms will be completed by the various parties.
10. Please note that there may be specific requirements depending on the asset owner or authorities, eg for tanks located beneath and near a public road, if tanks are within 3.7 m of the highway, the design and construction classification will be 3 and will require specific authority structural approval.

CDM background: note that under CDM2015 the following legal duties apply.

Client (commercial)

- To appoint the principal designer and principal contractor, ensuring they have the skills, knowledge, experience and organisational capability.
- Provide pre-construction information.

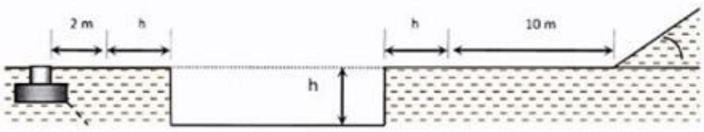
Principal designer

- Identify, collect and pass on pre-construction information between the parties.
- Co-ordinate all aspects of the design work.
- Liaise with the principal contractor regarding ongoing design work.
- Facilitate good communication between the appointees.
- Prepare and update the health and safety file.

Definition of zones of influence:

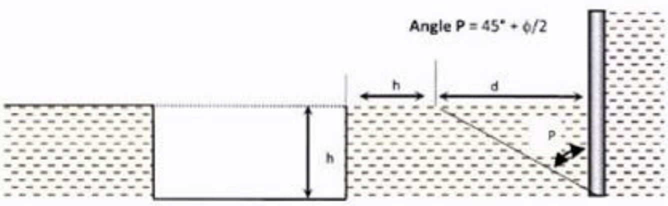
Slopes or stockpiles beyond h +10 m are not considered to be of influence.

Foundations or loaded/trafficked pavements beyond h + 2m are not considered to be of influence and considered to be **remote***



A slope is considered to be an incline greater than 10° and be greater than 1.5m in height

NB: Pile supported structures: zone of influence should be taken as x + h, where x = 5D for piles supporting vertical load only, x = 8D for piles supporting horizontal loads (D = pile diameter)



Excavation in front of retaining walls will need to take account of the passive zone supporting the wall. Angle P typically will vary from 55 to 65°. ϕ = the angle of shearing resistance of the soil. The distance d is dependent on the depth of the wall and the soil strength. Excavations beyond d + h are considered **remote***.

Notes on use of proforma

1. Each box in the following sections is awarded a score. All applicable boxes should be ticked, ie potentially more than one per section. The sum of the scores will determine the appropriate classification determined for the project assessed.
2. In general the greater the perceived risk the higher the score for the assessed element.
3. NB: Building Control Regulation specifies that a soakaway, domestic or otherwise, must be at least 5 m from any adjacent structure.

1. Type of site

Domestic single dwelling. (units less than 3 m³ capacity, project below notification requirements for CDM) ☐ Score = 0

Commercial application (CDM applies Part 3) ☒ Score = 10

2. Use

Soakaway ☐ Score = 5

Attenuation ☒ Score = ~~10~~ 5

Grey/rainwater storage ☐ Score = ~~10~~ 5

Other ☐ Score = ~~15~~
depends on risk

Specify ....

3. Pre-design/construction information held – In this example, it is assumed that all information is available from the Principal Designer. Score = 0.

The information is important for design. Geological mapping, a desk study and groundwater data are usually included in a basic site investigation along with information on soil types from boreholes, probe holes or trial pits.

The information listed is necessary to identify the design hazards (e.g., the overall site development plan will show if the tank is near foundations and the ground and groundwater information allows the pressure on the side of the tank to be estimated).

4. Topography/retaining walls/stockpiles/foundations – In this example, the site is on level ground. Score = 0.

If the tank is near anything that could impose additional load on the sides or top, give a score of 30. If the tank collapsed and could cause unacceptable movement or collapse of foundations, slopes, retaining walls, etc., then give a score of 60.

5. Installation development location and use – In this example, the tank is in a car park (general) with no height access restrictions. Score = 20.

Choose one of the locations/uses identified in the table on the proforma. Judgement will be required to assign the use of the site to one of the categories. The basic principle is that the greater the consequences of failure the higher the score.

6. Depth of installation In this example the tank is 2.4 m deep (i.e., between 1 m and 3 m to base). Score = 5.

In this example, the tank has greater than 1m cover and is subject to traffic. Score = 15.

The worst of the two scores is applied in the scoring system otherwise double counting can occur.

In this case, the worst score is given by the cover and traffic. Score = 15.

7. Construction phase – In this example, there is no construction access or stockpiles over the tank and an exclusion zone will be implemented. Score = 0.

If several of these factors apply, then use the worst-case value to determine the score to avoid doubling up.

Consider each site individually to assess if any other site-specific factors could affect the score.

Assessment total score - Add up the individual scores. For this example, Total = **50**.

Using **Table 3.2 C737** for this example the Site Classification is 1.

3. Pre design/construction information held

Information held by principal designer and distributed to the designer, supplier/manufacturer and principal contractor. Score = 0 for single domestic dwellings. Tick all boxes where info held. If <u>any</u> information missing Total score for <u>this</u> section = 35

Local knowledge/ geological mapping ☒	Basic ground investigation, confirmation of soil type (window samples + TPs) ☒	Ground water data/assessment ☒
Desk study ☒	Services information/search ☒	Overall site development plan ☒

4. Topography/retaining walls/stockpiles/foundations (<u>within</u> zone of influence)

Adjacent to sloping existing ground, embankments or temporary stockpiles ☐ Score = 30	Adjacent to existing or planned, structures retaining walls, piles or shallow foundations ☐ Score = 60	Adjacent to level ground (defined as h +10 m, see definition diagram) ☒ Score = 0

5. Installation development location and use

Residential garden (remote*) ☐ Score = 0	Pasture/woodland/parkland/ (remote*) landscaping ☐ Score = 0	Arable farmland (tractor/ harvester access) ☐ Score = 5
Residential driveway/play areas/sports field ☐ Score = 5	Car park (light use with height access restrictions) ☐ Score = 15	Car park general (no height access restrictions) ☒ Score = 20
HGV parks/low speed roads, installation **within** zone of influence ☐ Score = 30	Full highway loading installation **within** zone of influence ☐ Score = 80	Railway loading installation **within** zone of influence ☐ Score = 110

6. Depth of installation

Less than 1.0 m to base ☐ Score = 0	Cover less than 1.0 m and trafficked ☐ Score = 25
Between 1.0 and 3.0 m to base ☒ Score = 5	Cover greater than 1.0 m and trafficked ☒ Score = 15
Greater than 3.0 m to base ☐ Score = 20	Cover to units 0.3 m to 2.0 m landscaped ☐ Score = 10
	Cover greater than 2.0 m ☐ Score = 15

7. Construction phase (temporary works, TW)

TW stockpile/plant stored within zone of influences ☐ Score = 25	High ground water likely within excavation ☐ Score = 20
TW access/construction plant tracking over installation (excluding plant used in construction of the actual tank) ☐ Score = 20	Plant/materials exclusion zone implemented within zone of influence ☒ Score = 0
Use of mobile or tower cranes within zone of influence ☐ Score = 30	No provision for ground/rainwater removal, ie pumped sump ☐ Score = 15

Assessment total score 50

6.2 Results of the site classification and implications

In this example, the site is classified as Class 1 with the following implications:

- Undertake design checks for vertical distributed and concentrated loading.
- Check adequacy of cover over units to distribute wheel loads.
- Check uplift, if appropriate (for tanks below groundwater).
- Assess earth pressures using active pressure coefficient.
- Use standard test methods and data for the properties of the geocellular units.

These checks are explained in the following worked example.

In this example, the Class 1 requirements will mean that the design checks are completed by a competent building professional with relevant industry experience. An Incorporated or Chartered Engineer is to oversee the design checks. Drainage Design Consultants Limited (the company responsible for the design in this example) should confirm that these requirements have been met.

6.3 Generic classification system for routine sites

A generic classification system for different zones has been prepared for sites where the tank design will be routine and there are no special circumstances (i.e., the tanks are not unusually deep or shallow or are not within the zone of influence of slopes, buildings, etc.). The classification is provided in Table 1 of this guide. This is based on the following traffic zones (further information on the zones is provided in Appendix B of this guide).

A	Anywhere that vehicle access is not possible (e.g., due to fences or barriers, road layout or topography).
B	Anywhere that only cars can access due to physical constraints.
C	Anywhere that HGVs will only access as an "accidental load" (i.e., not regular HGV traffic, for example, vehicle overrun on a verge at the back of a footway).
D	Anywhere that is subject to limited HGV traffic at very low speed (<15 mph) such as fire tenders and refuse trucks.
E	Everywhere else (assumed to be subject to regular unrestricted HGV traffic). This category is split into three sub-categories depending on the type of HGV loading that is expected (E1 to E3). E1 is for areas where HGVs will be regular and moving at low speeds such as lorry parks and loading bays. E2 would cover some estate roads in residential developments and E3 would cover trunk roads and motorways. In the latter case in the running lanes of motorways (including the occasional hard shoulder on Smart Motorways), specific assessment of the special vehicle loads should be undertaken to the requirements of Highways England.

Table 1 Generic Classification

Traffic zone	General description	Type of site	Score	Use	Score	Information	Score	Topography	Score	Location	Score	Depth to base	Score	Cover (see note at base of table)	Score	Construction phase	Score	Classification Total score	Classification Class	Testing requirements	Recommended actions/roles (Table 3.2 C737)	Design requirements (Table 3.3 C737)	Checking requirements (Table 3.2 C737)
A	No vehicular access	Commercial	10	Attenuation	5	Assume all relevant information is available	0	Level ground	0	Equivalent to parkland	0	1 m to 3 m	5	0.3 m to 2 m landscaped	10	Assume some construction plant passing over	20	50	1	Long-term creep rupture and short-term tests (300 mm diameter and full plate)	Simple design calculations by competent building professional with relevant industry experience	Check units have sufficient strength to support vertical loads (distributed and concentrated). Check cover to units is sufficient to distribute concentrated loads and to prevent flotation. Assess earth and water pressure on sides using standard methods and assuming active earth pressure coefficients apply	Simple design checks to be undertaken by competent building professional. Independent check by another engineer who may be from the same team (Incorporated or Chartered Engineer to oversee checks)
B	Car access only	Commercial	10	Attenuation	5		0	Level ground	0	Equivalent to car park light use	15	1 m to 3 m	5	1 m to 2 m trafficked	15		20	70	1				
C	Accidental HGV access	Commercial	10	Attenuation	5		0	Level ground	0	Equivalent to car park general	20	1 m to 3 m	5	1 m to 2 m trafficked	15		20	75	1				
D	Limited HGV traffic at low speed	Commercial	10	Attenuation	5		0	Level ground	0	Low speed roads	30	1 m to 3 m	5	1 m to 2 m trafficked	15		20	85	2	Long-term creep rupture and short-term tests (300 mm diameter and full plate)	Design by Chartered Civil Engineer with 5 years 'post chartered' specialist experience in ground engineering	Check units as above. Consider allowable movements and assessment of manufacturer's data. Consider creep deformation. Detailed assessment of construction activities.	Design overseen by Chartered Civil Engineer with 5 years 'post chartered' specialist experience. Category 2 check by an Engineer who must be independent of the design team but can be from the same organisation
E1	Regular HGV traffic at low speeds	Commercial	10	Attenuation	5		0	Level ground	0	HGV park	30	1 m to 3 m	5	1 m to 2 m trafficked	15		20	85	2				
E2 and E3	All other locations. High speed HGV traffic	Commercial	10	Attenuation	5		0	Level ground	0	Equivalent to full highway loading	80	1 m to 3 m	5	1 m to 2 m trafficked	15		20	135	3	Long-term and short-term tests as above plus cyclic loading tests (fatigue test). Full-scale pavement tests if less than 1 m cover to tank	Design by Chartered Civil Engineer with Geotechnical Advisor status	As above plus assessment of fatigue and cyclic loading and detailed assessment of deformations. Numerical modelling required	Senior Specialist Geotechnical Engineer with Geotechnical Advisor status should be appointed to oversee design process, likely complex modelling and testing required. Category 3 check by an Engineer from a separate organisation to that of the designer.
NOTES:		Assume all locations are "commercial"		Assume attenuation is worst case. Note - there is no reason why attenuation is greater risk than soakaway so score for soakaway has been used				Assume for this first stage, level ground and outside zone of influence of walls, etc.						Assume >1 m but less than 2 m = 0. Not explicitly stated		Assume the tank is not below groundwater table		Assume tank is outside zone of influence of any structure etc. i.e. Zone 4		Assumes units are not prone to excessive bending or instability when subject to shear loads or other uneven loading (units assembled on site from plates require specific shear testing)			

7. Step 2: Develop the conceptual ground model

(Pages 78 - 82 C737)

The purpose of the conceptual ground model is to describe the tank installation and the surrounding ground. It will also include any slopes or nearby structures that will influence the design. The conceptual ground model forms the basis of the design analysis and calculations.

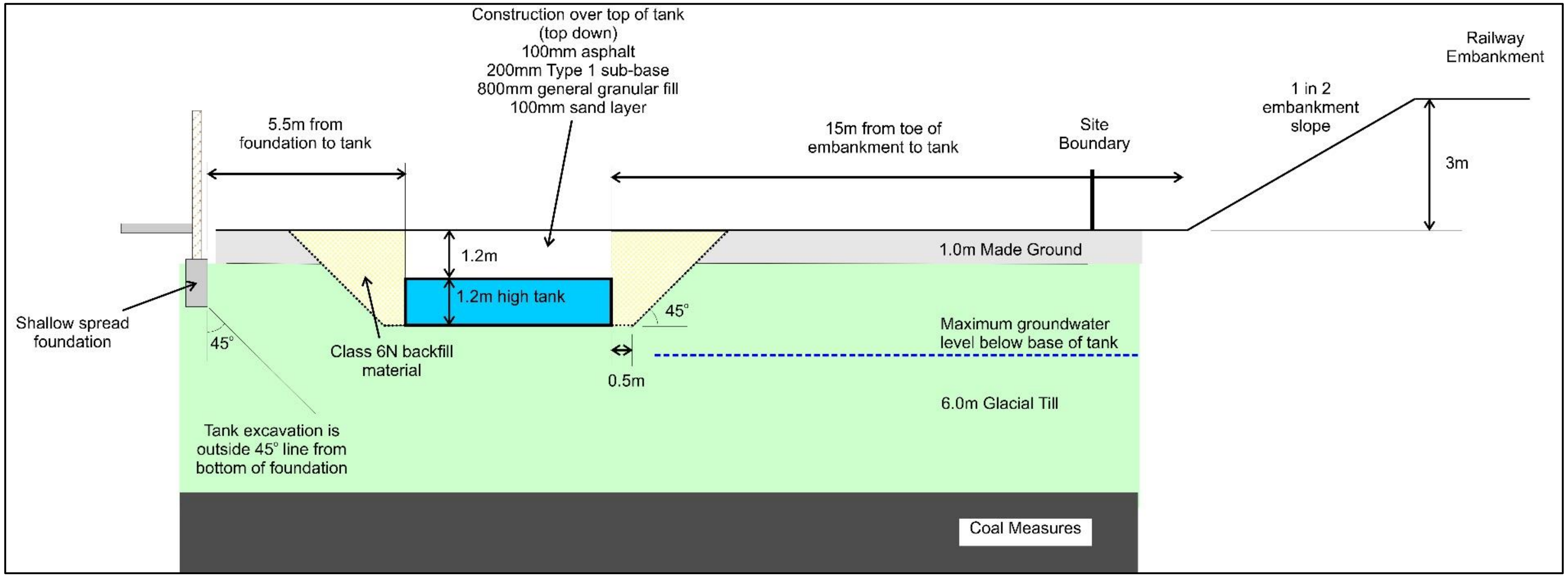
The best way to present the ground model is for the designer to draw up a cross-section of the proposed tank installation showing the tank, backfill details, excavation limits, backfill materials, nearby slopes or walls, etc. The properties of the tank installation and the surrounding ground should be summarised on the ground model.

The key items are:

- Ground level profile over and adjacent to tank.
- Depth of cover over top of tank.
- Depth to base of tank.
- Geological profile of ground around the tank.
- Soil or rock properties of the surrounding ground and proposed backfill.
- Extent of excavation for the tank.
- Strength and deformation properties of the proposed tank.
- Nearby structures, slopes or other features that may influence the design and performance of the tank.

The conceptual ground model for the site and tank being considered in this worked example is provided in Figure 2.

Figure 2 Example conceptual ground model



Ground properties

Stratum	Typical thickness	Unit weight	Effective angle of friction
Made Ground (medium dense black sandy GRAVEL of ash and clinker)	1.0m	18kN/m³	32°
Glacial Till (firm to stiff dark grey silty sandy CLAY with much fine to coarse gravel)	6.0m	20kN/m³	28°
Coal Measures (not investigated). Geological map indicates series of mudstone, siltstone, sandstone and coal seams. No workings	100m+	n/a	n/a
Class 6N backfill to Specification for Highway Works	--	18kN/m³	36°
Class 1 General granular fill to Specification for Highway Works	--	20kN/m³	32°

Manufacturer declared values for properties of geocellular tank

Unit	Mr Plastic Manufacturing Company Ltd, Waterbox 1	
	Vertical	Horizontal
Ultimate strength (short-term mean value)	440kN/m²	97kN/m²
Characteristic strength (long-term, 50 years)	124kN/m²	27kN/m²
Design strength (50 years)	83kN/m²	18kN/m²
Reference strength (20 years)	85.5kN/m²	18.5kN/m²

See *Product Evaluation Form* for further information (**C737 Page 151**)

8. Step 3: Determine characteristic loads and apply partial factors to give design loads

8.1 Loads

The following loads will be calculated:

Step 3.1: Vertical characteristic load from backfill and surcharge.

Step 3.2: Vertical characteristic traffic loading.

Step 3.3: Lateral characteristic load from earth pressure and groundwater.

Step 3.4: Lateral characteristic load from wheel loads adjacent to tank.

8.2 Step 3.1: Vertical characteristic load from backfill and surcharge

(Pages 80 - 82 C737)

The purpose of this step is to define the permanent loads from the backfill and any likely long-term surcharge (such as long-term piles of soil or other materials). This part of the calculation does not include surcharge loads that are transient and part of the traffic load assessment.

In this example, most of the fill over the tank is soil. The pavement layers (sub-base and asphalt) may have different unit weights to the soil backfill. However, in this case the pavement layers are thin in relation to the overall cover depth and so variations in unit weight will not make any significant difference to the applied load and a single value of 20 kN/m^3 is assumed for all the soil backfill.

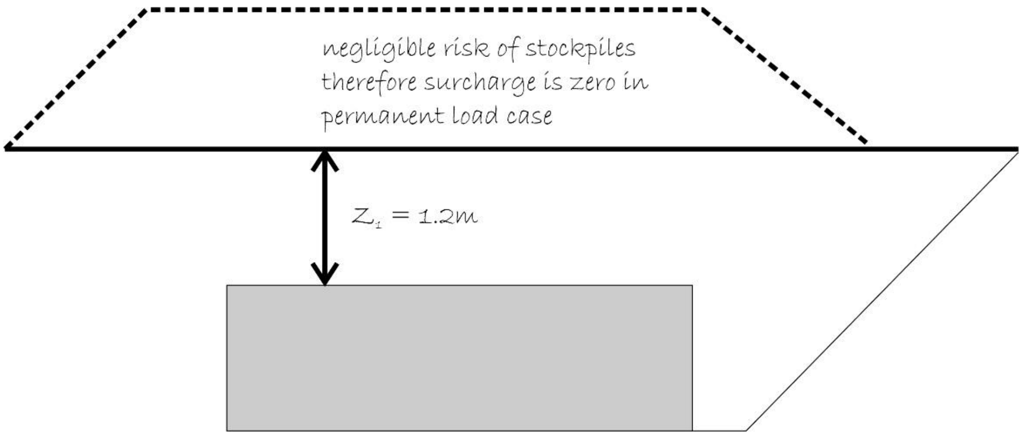
Where the depth of cover varies, two sets of calculations will be required using the maximum and minimum cover depths. The minimum cover depth gives the least distribution of concentrated loads such as wheel loads (and thus a higher transient load on the tank). The maximum cover depth gives the highest permanent load (and greater potential for creep failure) although the load from wheels will be lower because it is distributed over a greater area.

The unit weight of the fill material should be taken from **Table 5.4 C737** which gives typical values for various types of soil and materials. In this case, the tank will be covered by general granular fill which is mainly derived from ash and clinker excavated on site. It will be compacted so the value of unit weight from **Table 5.4 C737** for dense slag fill (20 kN/m^3) should be used.

Variations within the likely range of values for the unit weight of typical backfill materials will make little difference to the results. Values less than 19 kN/m^3 would need verification testing of fill material on site to make sure it is achieved. This is because only slight increases in permanent load can have a significant effect on the magnitude of creep deformations and time to failure.

	Project: BPF Towers	Page: 1
	Description: Example design	
	Designer: BPF Pipes Group	Date: Feb 2017

Characteristic load from backfill and surcharge (permanent)



Depth of fill over top of tank, $Z_1 = 1.2$ m

Unit weight of fill, $\gamma = 20$ kN / m³

Characteristic permanent distributed load, $Q_{ckP} = Z_1 \times \gamma$

$= 1.2 \times 20 = \underline{24 \text{ kN / m}^2}$

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8.2 Step 3.2: Vertical characteristic traffic loading

(Pages 83 - 86 C737)

The purpose of this step is to define the transient loads, which are typically those from traffic. Transient loads can be concentrated (e.g., wheel loads) or distributed (e.g., surcharges). In some cases (depending on the cover depth over the tank), the zone of influence of two wheels may overlap at the top of the tank. This increases the pressure on top of the tank in the zone of overlap.

The wheel load (which is half the axle load) and surcharge loads may be taken from Appendix C of this guide. The approach described in Appendix C is based on the guidance in C737 but has been expanded to provide a greater range of load scenarios. In this example, the design is for a tank in a general car park which is equivalent to Load Class C.

In this example, the wheel load, Q_w, from Appendix C is 100 kN (Zone C – Car parks without barriers or anywhere HGVs will only access as an accidental load and not regularly).

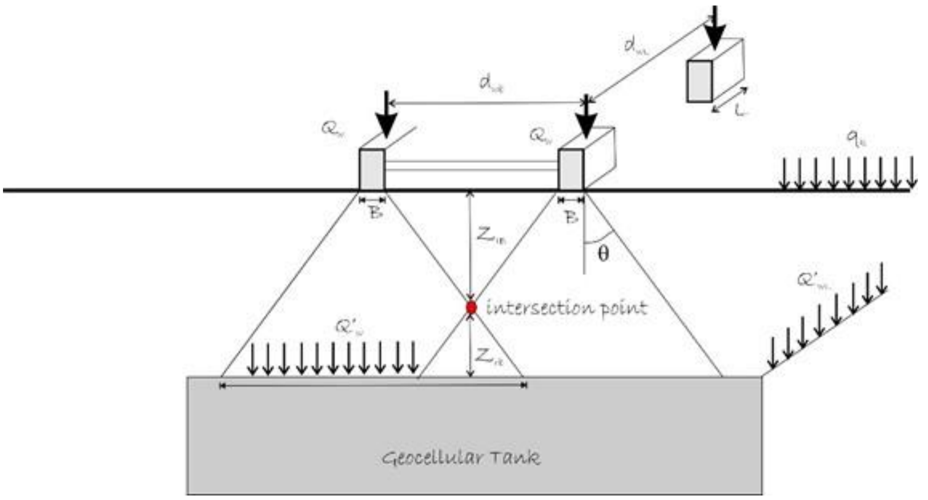
Guidance on appropriate values for the dynamic amplification factor, adjustment factor and overload factor are given in Appendix C. For Zone C, the dynamic amplification factor and overload factor = 1.0 and the adjustment factor is 0.8.

The plan of the wheel layout is given in **Figure 5.5(b) C737** and the spacing of wheels on an axle and between axles can be taken from that.

In this example, because the tank is covered mainly by general soil fill, a load spread angle of 26.6° is used. A more detailed analysis of load spread following the guidance on **Page 88 C737** may reduce the loading on the tank slightly.

The first calculation, shown on the adjacent page, is to determine the equivalent width, B' and L', of load application at the top of the tank (after load spread through the soil). These values will be used to determine the pressure applied to the top of the tank.

Characteristic load from traffic (transient)



Input Values:

Characteristic surcharge pressure for traffic, $g_K = 5.5$ kN / m²

Wheel load, $Q_W = 100$ kN

Wheel contact width, $B = 0.4$ m

Wheel contact length, $L = 0.4$ m

Dynamic amplification factor, **DAF** = 1.0

Adjustment factor = 0.8

Overload factor, **OLF** = 1.0

Distance between centreline of adjacent axles, $d_{WL} = 1.2$ m

Distance between centreline of wheels on one axle, $d_{WB} = 2.0$ m

Load spread angle through pavement and fill, $\theta = 26.6°$

Calculate:

Extent of load spread at top of tank

Equivalent width $B' = (2 \times Z_1 \times \tan\theta) + B$

$B' = (2 \times 1.2 \times \tan 26.6°) + 0.4 = \underline{1.6\ m}$

Equivalent length $L' = (2 \times Z_1 \times \tan\theta) + L$

$L' = (2 \times 1.2 \times \tan 26.6°) + 0.4 = \underline{1.6\ m}$

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The calculation shown on the adjacent page is to determine the depth to the intersection point of the load spread lines from adjacent wheels. The depth from the intersection point to the top of the tank is then calculated. This is all based on simple geometrical analysis and allows the zone of overlap to be determined.

If the point of intersection is above the tank, then the applied pressure in the overlap area is twice that from a single wheel.

The load applied to the top of the tank from a single wheel is based on the spread angle and the depth to the top of the tank.

	Project: BPF Towers	**Page:** 3
	Description: Example design	
	Designer: BPF Pipes Group	**Date:** Feb 2017

Depth of intersection point between wheels, Z_{IB}

By simple geometry

$$Z_{IB} = 0.5\ \frac{(d_{WB}-B)}{TAN\theta} = 0.5\ \frac{(2.0-0.4)}{TAN\ 26.6°} = \underline{1.6\ m}$$

Depth of intersection point between adjacent axles, Z_{IL}

$$Z_{IL} = 0.5\ \frac{(d_{WL}-L)}{TAN\theta} = 0.5\ \frac{(1.2-0.4)}{TAN\ 26.6°} = \underline{0.8\ m}$$

Overlap of pressure bulbs between wheels

Depth from intersection point to top of tank, Z_{RB}

$Z_{RB} = Z_1 - Z_{IB} = 1.2 - 1.6 = -0.4$ m (i.e. no overlap at top of tank)

Overlap of pressure bulbs = $\underline{0\ m}$

Overlap of pressure bulbs between adjacent axles

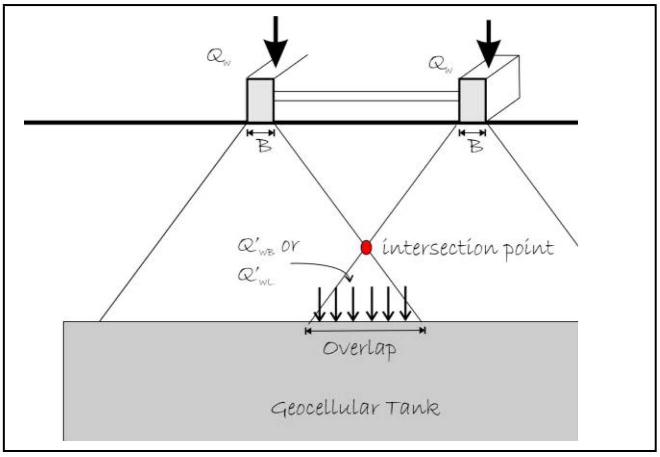
Depth from intersection point to top of tank, Z_{RL}

$Z_{RL} = Z_1 - Z_{IL} = 1.2 - 0.8 = \underline{0.4\ m}$

By simple geometry

Overlap = 2 x Z_{RL} x TANθ

= 2 x 0.4 x TAN 26.6° = $\underline{0.4\ m}$



	Checker: BPF Pipes Group	Date: 8/03/2017

The calculation shown on the adjacent page uses the load spread and overlap from the previous sheets to calculate the wheel load on the tank for a single wheel and in the overlap zone.

The total characteristic load from traffic is the sum of the load applied at the top of the tank from the wheel loads plus the transient surcharge load.

	Project: BPF Towers	**Page:** 4
	Description: Example design	
	Designer: BPF Pipes Group	**Date:** Feb 2017

<u>Wheel load on tank, no overlap, Q'_W</u>

$$Q'_W = \frac{Q_W \, x \, DAF \, x \, Adjustment \; Factor \; x \; OLF}{B' \, x \, L'} = \frac{100 \; x \; 1.0 \; x \; 0.8 \; x \; 1.0}{1.6 \; x \; 1.6} = \underline{31.25 \; kN / m^2}$$

<u>Wheel load on tank, zone of overlap adjacent to axles, Q'_{WL}</u>

$Q'_{WL} = 2 \times Q'_W = 2 \times 31.25 = \underline{62.5 \; kN / m^2}$

In this case, Q'_{WB} is the same as Q'_W because there is no overlap in that direction.

<u>Total characteristic load from traffic, Q_{ckT}</u>

Q_{ckT} = Wheel load + surcharge load

Use maximum value of wheel load from Q'_W, Q'_{WL} and Q'_{WB}

$Q_{ckT} = (62.5 + 5.5) \; kN / m^2 = \underline{68.0 \; kN / m^2}$

	Checker: BPF Pipes Group	Date: 8/03/2017

8.3 Step 3.3: Lateral characteristic load from earth pressure and groundwater
(Pages 89 - 91 C737)

The purpose of this step is to define the permanent lateral loads that act horizontally on the side of the units (normally the earth and groundwater pressure). Additional pressure from transient loads such as wheels and/or surcharges is calculated separately.

The design for lateral loading is based on the maximum pressure that will occur at the bottom of the tank. The characteristic value is Q_{ckPL}.

In this example, the pressure is derived from earth pressure only using the depth of 2.4 m. This is because groundwater is below the base of the tank so there is no groundwater pressure on the side of the tank. If groundwater is above the base of the tank, the water pressure should be added to the earth pressure (calculated using submerged density below the water table). If it is considered likely that groundwater could accumulate in the backfill around the tank over time (for example, in a tank in clay that does not have a route for infiltrating water to seep away), then an allowance for groundwater pressure should be assumed. For online tanks, water can usually seep away along the bedding to the outlet pipe.

The earth pressure is calculated using the angle of friction, ϕ', of the soil or backfill around the tank. If the failure plane for the active wedge is through the granular backfill, then the ϕ' for that material should be used. This typically occurs where there is a wide working space around the tank and a battered slope to the excavation. Otherwise use the ϕ' for the surrounding soil, typically where there is a narrow working space and a steep or vertical wall to the excavation. This is explained in **Figure 5.14 C737**. In this case, the diagram on the adjacent page shows the failure plane is through the Class 6N material and, therefore, $\phi' = 36°$.

The following earth pressure coefficients are suggested in C737:

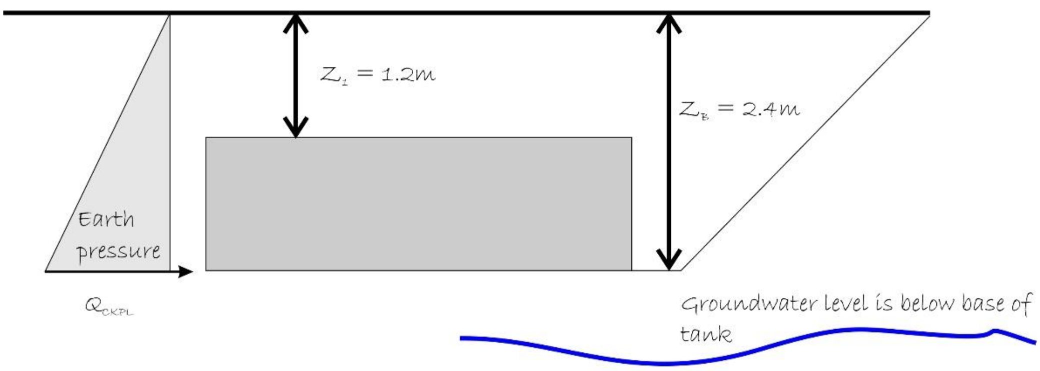
- Tank depth to base up to 3 m, active pressure coefficient, K_a.
- Tank depth to base between 3 m and 4 m, use average of active and at rest coefficients = $(K_a + K_o)/2$.
- Tank depth to base greater than 4 m, earth pressure coefficient at rest, K_o.

This is explained in more detail on **Page 93 C737**.

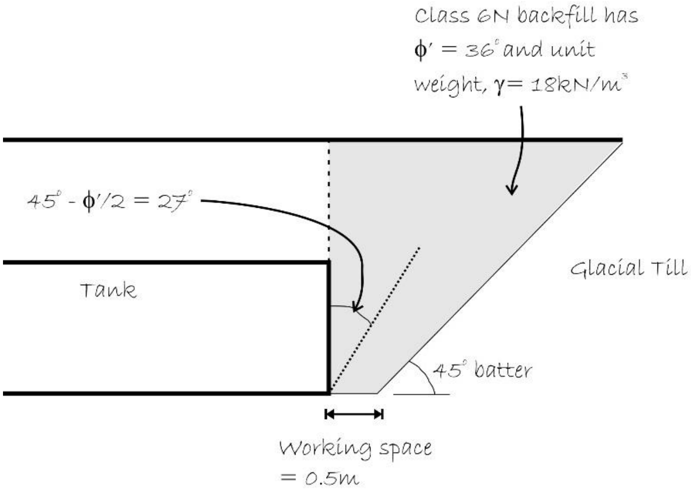
The equations to calculate K_o and K_a, along with those for calculating the earth and groundwater pressure, are provided in **Section 5.3.4.3, Pages 89 and 90, C737**. These are standard geotechnical equations that are widely used in retaining wall design. In this example, the depth is less than 3 m and so K_a is used.

In this case, the Class 6N backfill will be compacted with a small vibrating plate compactor. **This needs to be communicated to the contractor in the geotechnical design report including the maximum allowable compaction plant (load) assumed in the design.** Experience from the past 20 years has shown that this approach does not induce excessive compaction pressures on the tanks. However, if required a specific analysis for compaction pressure can be completed following the guidance on **Pages 90 and 91 C737** (compaction induced pressures).

<u>Characteristic lateral load from earth pressure and groundwater, Q_{ckPL}</u>



Depth of base of tank = Z_B = 2.4 m
Effective angle of friction of backfill, ϕ' = 36 °
Page 94 of C737, Fig 5.14



Active wedge forms at $45° - \frac{\phi'}{2} = 45° - \frac{36}{2}$ = <u>27°</u>
Active wedge forms in Class 6N backfill material
Therefore, use ϕ' = 36 ° in design
Φ'_{BD} = 36 °
Page 93 of C737, depth is less than 3 m so use K_a, active pressure coefficient

$$K_a = \frac{1 - SIN\phi'}{1 + SIN\,\phi'} = \frac{1 - SIN\ 36°}{1 + SIN\ 36°} = 0.26$$

$Q_{ckPL} = K_a \times \gamma \times Z_B$ = 0.26 x 18 x 2.4 = <u>11.23 kN / m²</u>

8.4 Step 3.4: Lateral characteristic load from wheel loads adjacent to tank

(Pages 92 - 93 C737)

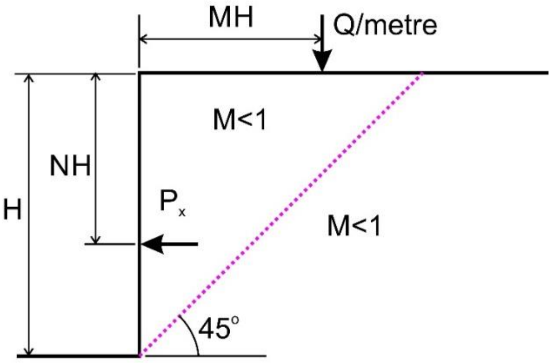
The purpose of this step is to define the horizontal loads on the side of the tank that are caused by vehicle wheels located adjacent to the tank. The load is transmitted through the soil onto the side of the tank.

In this example, the approach described by Georgiadis and Anagnostopoulos (1998)[1] is used. This is explained in **Figure 5.11(b) C737**. For simplicity, the wheel load is treated as a strip load equal to the width of a wheel and is assumed to be continuous along the wall. This is conservative but not excessively so and simplifies the analysis.

The applied pressure determined using this approach will vary with distance of the wheel from the tank. The critical distance that results in the maximum pressure at the top of the tank has first to be determined, prior to completing the Georgiadis and Anagnostopoulos analysis.

To do this the pressure distribution from the wheel is assumed to be a line load (or knife edge load). In this example, it has been derived from the wheel load using **Equation 5.11 from Page 92 C737**. The applied pressure is calculated for each distance from the back of the wall using a Boussinesq stress analysis (see Figure 3 and the equation below). This makes no allowance for the soil properties. It does, however, give an indication of the likely dissipation of lateral loads from the wheel in the soil above the top of the tank wall.

Figure 3 Derivation of pressure on side of tank from line load



$$P_x = \frac{2Q}{\pi H} \cdot \frac{M^2 N}{(M^2 + N^2)^2}$$

The graph on the adjacent page has been derived using this approach, assuming the wheel load in this example is 100 kN/m² applied over a 400 mm by 400 mm contact area (as defined for Zone C in Appendix B of this guide). The load is multiplied by the appropriate adjustment, dynamic and

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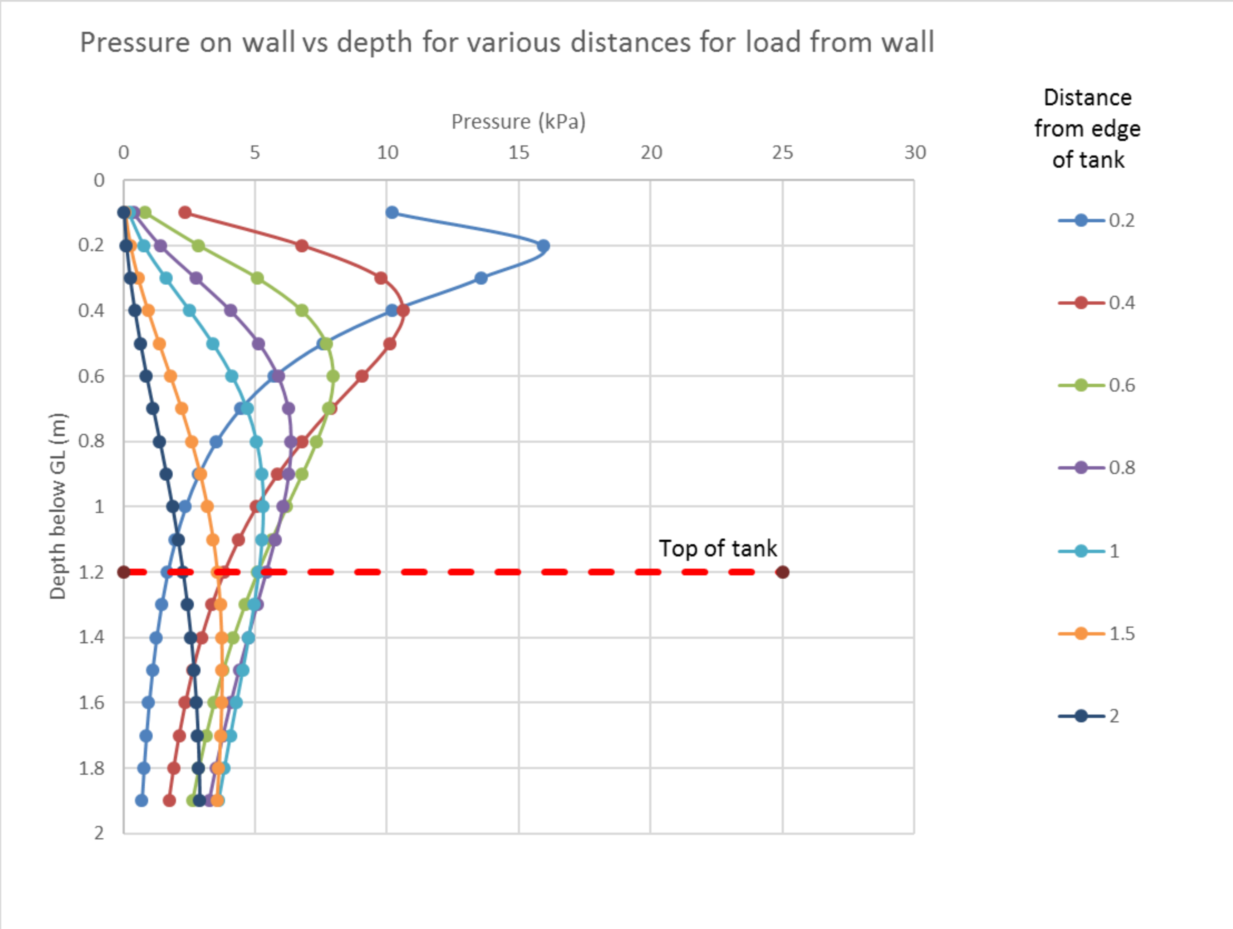
[1] Georgiadis M and Anagnostopoulos C (1998). Lateral Pressure on Sheet Pile Walls due to Strip Load. Journal of Geotechnical and Geoenvironmental Engineering Vol 124 Issue 1 January 1998. ASCE pp95 – 98.

overload factors from the previous sheets. The critical distance, A, at which the greatest pressure is applied (at the level of the top of the tank) can then be determined.

The graph for this example is shown below. It is used to determine the critical distance for the wheel load from the tank for the design cover depth. In this example, the top of the tank is at 1.2 m depth and the maximum pressure occurs when the wheel is 0.8 m from the tank (i.e., A= 0.8 m). This distance, A, must not exceed the cover depth of the tank.

In the equation above, the factor 2 allows for a flexible wall as explained in *Foundation Analysis and Design* (J E Bowles, 4th Edition, McGraw-Hill International, 1998). Geocellular tanks are considered to be flexible.

Figure 4 Variation of pressure on side of tank from wheel load for this example



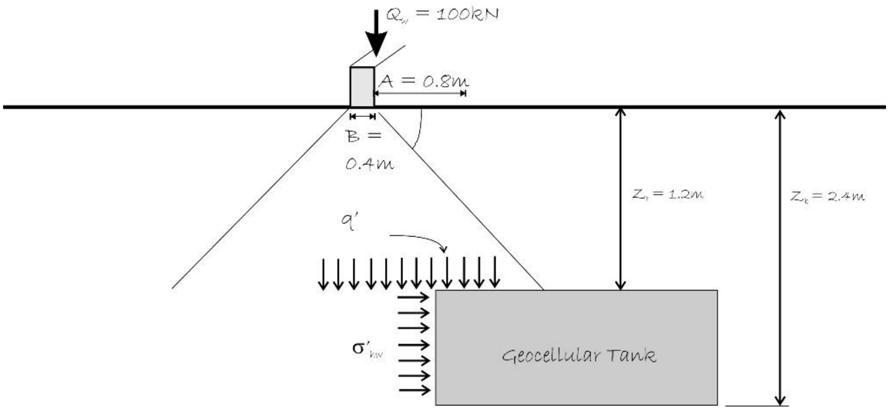
Once the distance, A, has been determined using the Boussinesq analysis, the pressure on the side of the tanks is calculated using Georgiadis and Anagnostopoulos (1998) as shown on the adjacent page.

In this example, the friction between the wall and the backfill is taken as zero. This is conservative and if there is sufficient information about the interface friction for the geotextile or geomembrane that is to be used, then an allowance may be made for friction.

In this case, the active earth pressure coefficient, K_a, is used as described previously. See the previous permanent lateral load calculations (from earth pressure and groundwater) for a discussion about the appropriate earth pressure coefficient to use.

Characteristic load (lateral) from traffic, Q_{ckTL}

Use method of Georgiadis and Anagnostopoulos (1998), **Figure 5.11 of C737**



Dynamic amplification factor and overload factor = 1.0 (see vertical load calculations). Adjustment factor = 0.8 (see vertical load calculations)

Convert concentrated wheel load to strip load

Equivalent strip load is calculated using **Eq 5.11, Page 92 of C737**

$Q_L = \frac{Q_W}{2\,A+L}$ Q_W is multiplied by factors above

$Q_L = \frac{100\ x\ 1.0\ x\ 1.0\ x\ 0.8}{(2\ x\ 0.8)+0.4}$ = 40 kN / m (in this example, the term L from C737 is the same as B in the diagram above – i.e. the width of the wheel, 0.4m).

Calculate pressure on back of wall

In this example, assume friction between wall and tank is zero, $\delta = 0°$

Characteristic pressure from wheel **σ'_{hw}**

σ'_{hw} = K_a x cos δq' = 0.26 x 1.0 x 8 = 2.08 kN / m²

where **q'** $= Q_L\ (\frac{B}{B+2A}) = 40\ x\ (\frac{0.4}{0.4+(2\ x\ 0.8)})$ = 8 kN / m

On the adjacent page, the pressure from the transient surcharge (traffic surcharge load) is calculated. The equation used is from standard earth pressure theory:

Lateral pressure = surcharge pressure x earth pressure coefficient.

The maximum value of pressure from either the wheel load (previous sheet) or the transient surcharge (this sheet) is used in the design to estimate pressure on the side of the tank from traffic.

There is normally no need to carry out a specific analysis of braking forces from vehicles approaching a tank in a direction that is perpendicular to the side (as suggested on **Page 89, C737**). The advice in C737 is based on the design of bridge decks and abutments where such loads are transferred into the structure. It is highly conservative when applied to geocellular tanks buried in the ground. Appendix D provides evidence to demonstrate that analysing braking forces from vehicles moving towards a tank is not appropriate where the cover over tanks is greater than 0.6 m in car parks and 1m where HGVs are travelling.

	Project: BPF Towers	Page: 7
	Description: Example design	
	Designer: BPF Pipes Group	Date: Feb 2017

Maximum pressure on tank from traffic surcharge load

Surcharge to allow for traffic is 5.5 kN / m^2

Pressure on tank due to surcharge q'_{hsur}

q'_{hsur} = surcharge pressure x earth pressure coefficient

= 5.5 x 0.26 = 1.43 kN / m^2

Use maximum of pressure calculated for concentrated wheel load or surcharge

Characteristic lateral load due to traffic

Q_{ckTL} = 2.08 kN / m^2

	Checker: BPF Pipes Group	Date: 8/03/2017

8.5 Step 3.5: Partial factors of safety for loads and soil properties
(Pages 99 - 100 C737)

Partial factors applied to loads

The purpose of this step is to determine the appropriate partial factors of safety that should be applied to the characteristic loads or soil properties to arrive at design loads. The partial factors applied to the properties of the geocellular units are explained in Section 9 of this guide.

Load factors for ultimate and serviceability states are provided in **Table 5.9 C737** and those used in this example are shown on the adjacent page. For lateral loads, Combination 1 in EC7 is assumed for routine design to assess the resistance of the tanks to lateral pressure. Combination 2 would be applicable for global stability checks such as slope stability analysis, where this is required. Note that there may be instances where Combination 2 in EC7 gives the worst-case pressure on the tank (e.g., if there are large variable surcharge loads and the retained soil has a high angle of friction).

Unfavourable loads are those that adversely affect the tank (e.g., the permanent load from the weight of soil on top of the tank, traffic loads and the pressure from earth on the sides of the tank).

Favourable loads are those that are beneficial to the stability being assessed. The most common is the weight of soil on top of the tank when used in assessment of uplift due to buoyancy of a tank below groundwater.

Note: Row 15 – Table 5.9, Equation 5.12 in C737 includes a dynamic load factor taken from Table 5.10 C737. This is doubling up on the DAF used in determining the characteristic loads. The LM1 loads taken from the Eurocodes (National Annexe to BS EN 1991-2: 2003 Traffic Loads on Bridges) already include a dynamic allowance. An additional DAF is not applied in this example.

The site importance factor is taken as 1 in this example because the site classification is 1.

Hydrostatic load acting vertically on top of units should be considered a permanent load. However, it is strongly recommended that tanks are designed to avoid being completely submerged below groundwater. This approach increases the risks of leakage of groundwater into the tank as well as structural failure. Completely submerged tanks should be classified as Class 3.

Partial factors applied to soil properties

Table 5.12 C737 gives the partial factors to be applied to soil properties (i.e., to the strength parameters of the soil).

For this assessment (Combination 1 in EC7) the factors are 1.0 in all cases. Combination 1 is the load scenario used for routine analysis. Combination 2 would be applicable for global stability checks such as slope stability analysis.

	Project: BPF Towers	Page: 8
	Description: Example design	
	Designer: BPF Pipes Group	Date: Feb 2017

Design loads (vertical and lateral)

Partial factors – Load (Table 5.9 of C737)
Permanent unfavourable action = 1.35
(vertical and lateral Combination 1) γ_{LFP}

Variable action unfavourable = 1.50
(vertical and lateral combination 1) γ_{FLFT}

Site importance factor γ_{SF} = 1.0 (site classification of 1)
= 1.0 for accidental loading

Partial factors on soil properties (Combination 1 in EC7) (Table 5.12 of C737)
On friction angle = 1.0
On cohesion = 1.0

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8.6 Step 3.6: Design vertical loads

The purpose of this step is to derive the design vertical loads using the characteristic loads and partial factors of safety from the previous calculation sheets.

Design loads = characteristic loads x partial factor of safety.

The calculations for this example are shown on the adjacent page for both permanent and variable loads.

	Project: BPF Towers	Page: 9
	Description: Example design	
	Designer: BPF Pipes Group	Date: Feb 2017

Design loads (vertical and lateral)

Design vertical loads

Design load = characteristic load x γ x site importance factor

Design vertical permanent load = characteristic load from backfill and surcharge x γ_{LFP} x γ_{SF} = 2.4 x 1.35 x 1.0 = 32.4 kN / m^2

Design vertical variable load = characteristic load from traffic x γ_{LFT} x γ_{SF} =68.0 x 1.50 x 1.0 = 102.0 kN / m^2

	Checker: BPF Pipes Group	Date: 8/03/2017

8.7 Step 3.7: Design lateral loads

(Pages 89 - 93 C737)

The purpose of this step is to derive the design lateral loads using the characteristic loads and partial factors of safety from the previous calculation sheets.

> Design loads = characteristic loads x partial factor of safety x lateral load reduction factor (LRF).

The calculations for this example are shown on the adjacent page for both permanent and variable loads.

The lateral load reduction factor (LRF) is to allow for arching around the tank. It is only applied to earth pressures and NOT to groundwater pressure. Using the LRF may not be applicable where excavations for tanks are within the global critical shear surface for adjacent slopes or foundations.

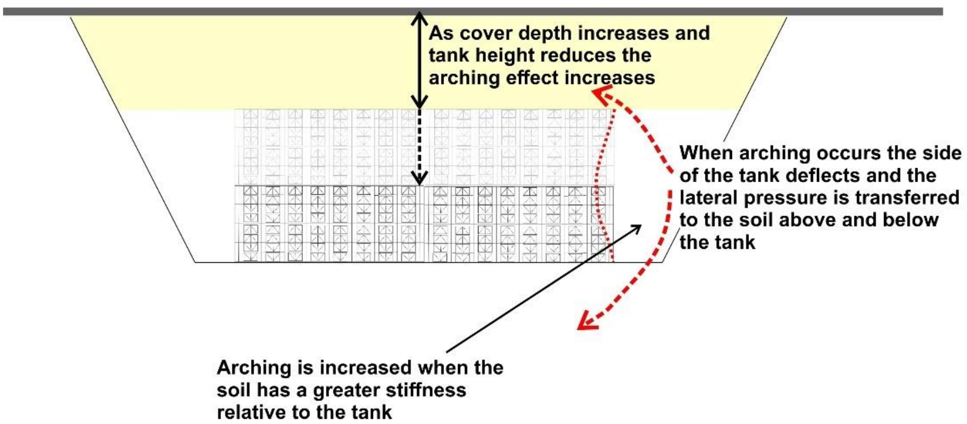
C737 suggests that the maximum lateral pressure on the side of a geocellular tank that occurs in practice may be less than that predicted by earth pressure theory because of arching in the soil (**Section 2.6.2 C737**). Arching is where the tank flexes and the pressure from the soil is transferred to the soil above and below the tank. The two main factors that affect whether arching will occur are the ratio of cover depth to tank height and the ratio of tank lateral stiffness to soil stiffness (see Figure 5 below).

The evidence in Appendix E of this guide indicates that at present, a conservative approach can be used to reduce the lateral pressure by 30% from the values predicted by Rankine earth pressure theory and those from the analysis of wheel loads following C737. The reduction can be applied to the maximum pressure calculated at the base of the tank when the following limiting conditions are met:

- The cover height to tank height ratio must be 0.48 or greater. This must be maintained where services pass over the top of tank.
- Soil to tank stiffness ratio must be 1.0 or greater (including the backfill over the top of the tank).
- Appropriate measures are put in place to prevent accidental excavation of the cover soils in locations that would impair the arching effect.

Further refinement and verification of the finite element model may allow greater reductions to be applied in a wider range of conditions.

Figure 5 Arching around a geocellular tank



Design lateral loads

The tank cover depth is 1.2 m and the tank height is 1.2 m. Therefore, the cover depth to tank height ratio = 1.0. This is greater than 0.48 and the reduction factor can be applied.

The failure wedge is in the Class 6N backfill. This will be much stiffer than the tank and the soil tank stiffness ratio will be greater than 1.0. Therefore, the reduction factor can be applied.

The lateral earth pressure can be reduced by 30% (i.e., load reduction factor = 0.7).

Design lateral permanent load
= (characteristic earth pressure x LRF + groundwater) x γ_{LFP} x γ_{SF}
= (11.23 x 0.7 + 0) x 1.35 x 1.0 = 10.61 kN / m^2

Design lateral transient load
= characteristic lateral pressure from traffic x γ_{LFT} x γ_{SF} x LRF
= 2.08 x 1.5 x 1.0 x 0.7 = 2.18 kN / m^2

9. Step 4: Determine characteristic strength and apply partial factors to determine design properties

(Pages 76 -78 C737)

9.1 Strength data

The characteristic strength and design strength would normally be declared by the supplier of the tank on the *Product Evaluation Form* (**Page 151 C737**). The form for this example is provided in Table 2, Section 9.5 of this worked example.

The process to be followed by the supplier of the tank to determine the properties is shown in **Figure 4.9 C737**. Currently there are no standardised test methods. Work is ongoing to develop European test standards but this is not likely to cover some of the tests discussed in C737 such as yield tests and fatigue (cyclic load) tests. More detailed advice on the current test regimes and how suppliers can provide data for design is provided in Section 12.2 of this guide.

At the time of publication of this guide, most units currently on the market have strength data that is based on tests that have been completed using the approach described in C680. Therefore, this example uses the data that is commonly available for most geocellular units. The short-term tests have been completed using a failure time of 10 minutes. **This is an interim process (also used by current BBA certificates) that should be followed until the information required for assessing the strength fully in accordance with C737 is published by manufacturers. Once European or UK Standard test methods are published, these should be adopted for testing the units.**

9.2 Step 4.1: Partial material factors of safety

(Pages 77 and 78 C737)

The purpose of this step is to show how a supplier would derive the partial factors of safety to be applied to the properties of the geocellular units. In the example on the adjacent page, the partial factor for the long-term creep strength is derived.

The partial factor for the geocellular unit properties is made up of many sub-factors that depend on the manufacturing process, variability of unit, extrapolation of test data, differences between laboratory and field performance, global influences (e.g., stacking units) and tolerance to construction damage.

The factors for this example are given on the adjacent page and are taken from **Table 5.2 C737**.

For this example:

- The units have creep test lab data with a maximum duration of 5,000 hours.
- Extrapolation of the lab test data from 5,000 hours to 50 years design life would lead to a higher factor of safety to allow for the uncertainty. However, it is assumed in this example that the units have a current BBA certificate and have been widely used for over 15 years at similar cover depths and vehicle loadings to the proposed installation and the supplier has provided robust evidence that no creep failure or excessive deflection has occurred over that time. **(Note the earliest installation of geocellular tanks in the U.K. was in the early 1990's)**.
- Although not a specific creep test, this information provides further evidence of the creep performance of the units and reduces the uncertainty in the extrapolation of the creep data to obtain a long-term strength. Therefore, the designer has used judgement to assess that a creep test equivalent duration of 10,000 hours can be adopted for deriving the partial factor of safety to be applied to the long-term strength to allow for uncertainty.

- Specific advice on a suitable factor of safety for extrapolation can be obtained from the manufacturer. It is envisaged that once specific tests standards are in place that longer creep test durations will remove the need for this approach to be used.
- The design life is 50 years.

In this example, the units are injection moulded units that are manufactured as two pieces. The units have been in use for over 15 years with no reported failures (caused by inadequate test data). Therefore, PF3 is assumed to equal 1.0.

The calculated partial factor should not be less than the minimum value of 1.5 quoted in C737.

The partial factor to be applied to the short-term strength in this example is derived in the same way. All the sub-factors are the same as for the long-term except PF2. For this factor, the same approach is used but the creep test duration is replaced with the number of load cycles completed in fatigue tests (or, where appropriate, the equivalent service duration at similar cover depths and vehicle loading to the proposed installation).

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	Project: BPF Towers	Page: 11
	Description: Example design	
	Designer: BPF Pipes Group	Date: Feb 2017

Partial material factors of safety

Partial factors PF_1 to PF_5 (**Table 5.2 of C737**)
Units are factory produced in one moulding, $PF_1 = 1.0$

Extrapolation of creep data
Maximum test duration of WaterBox 1 = 5,000 hours
However, units have been used for over 15 years with no reported failures, therefore, say creep test date is equivalent to 10,000 hours

$PF_2 = 1.2^r$ where $r = \log \frac{t_d}{t_{m2}}$

t_d = design life = 50 years = 438,000 hours
t_{m2} = creep test duration = 10,000 hours

$r = \log \frac{438000}{10000} = 1.64 \quad PF_2 = 1.2^{1.64} = 1.35$

Laboratory and mobilised strength
$PF_3 = 1.0$ (The evidence from the supplier shows that the laboratory test data is a reasonable indicator of the mobilised strength of the units when installed. Units have been in use for over 5 years with no known problems, use 1.0)
Global behaviour
$PF_4 = 1.0$ (The evidence from the supplier shows that there is no unusual global behaviour. Units have been in use for over 5 years with no known problems)
Damage during construction $PF_5 = 1.05$
Total material factor $\gamma_m = PF_1 \times PF_2 \times PF_3 \times PF_4 \times PF_5$
$\gamma_m = 1.0 \times 1.35 \times 1.0 \times 1.0 \times 1.05 = 1.42$
Minimum value = 1.5 for permanent works

	Checker: BPF Pipes Group	Date: 8/03/2017

9.3 Step 4.2: Design strengths

The purpose of this step is to derive the design strength (short-term and long-term).

The characteristic strength is divided by the appropriate partial factor as shown on the adjacent page.

	Project: BPF Towers	**Page:** 12
	Description: Example design	
	Designer: BPF Pipes Group	**Date:** Feb 2017

<u>Design strength</u>

Design strength = $\frac{\textit{Characteristic strength}}{\textit{Material partial factor } \gamma_m}$

Characteristic short- and long-term strength in the vertical and lateral direction for the WaterBox 1 are declared by the supplier on the Product Evaluation Form (Table 2 in Section 9.5 of this worked example).

Design vertical short-term strength, $P_{DS} = \frac{P_{CKS}}{\gamma_{ms}} = \frac{290}{1.5} = \underline{193.3 \text{ kN / m}^2}$

Design vertical long-term strength, $P_{DL} = \frac{P_{CKL}}{\gamma_{ms}} = \frac{124}{1.5} = \underline{82.7 \text{ kN / m}^2}$

Design lateral short-term strength, $P_{DSL} = \frac{P_{CKSL}}{\gamma_{ms}} = \frac{64}{1.5} = \underline{42.7 \text{ kN / m}^2}$

Design lateral long-term strength, $P_{DLL} = \frac{P_{CKLL}}{\gamma_{ms}} = \frac{27}{1.5} = \underline{18.0 \text{ kN / m}^2}$

	Checker: BPF Pipes Group	Date: 8/03/2017

9.4 Step 4.3: Product Evaluation Form

(Page 151 C737)

In this worked example, the geocellular units to be used are manufactured by Mr Plastic Manufacturing Company Limited. WaterBox 1 units will be supplied. Data supplied by the company on the *Product Evaluation Form* is shown on the adjacent page.

Testing and confirmation checklist – this part of the form shows the data that has been supplied by Mr Plastic Manufacturing Company Limited, given in Table 2 of this guide for this worked example. Note that professional indemnity insurance (PI) is not required for this example as Mr Plastic Manufacturing Company Limited is not contractually employed to provide design services. For schemes where the manufacturer/supplier is employed to provide the design, then PI is likely to be required. This information is required to allow the approach described in this guide to be used for design.

The porosity of the units in this example is 95%. Porosity is used in storage volume calculations. This value is placed in the box on the form labelled "Void Ratio". Note that void ratio is different to porosity (***SuDS Manual* 2015, Page 659**).

Porosity = volume of voids/total volume of material.

Voids ratio = porosity/(1 – porosity).

In the example *Product Evaluation Form*, the unit strength parameters are defined as follows:

- The ultimate strength is the mean value of the short-term strength derived using the laboratory test methods that are described in C680 and are used by BBA for most current certificates.
- Characteristic strength is the creep strength for the design life of the project – in this example 50 years.
- Design strength is the factored characteristic strength for the design life of the project – in this example 50 years.
- Reference strength is the creep strength for a design life of 20 years. The form in C737 incorrectly indicates that this is 50 years (to the right of the boxes) but the text in the main body of C737 makes it clear it should be 20 years.

The characteristic long-term or creep strength in this example has been derived by assuming the coefficient of variation for the short-term tests is the same as that for the creep tests. This has been shown by test data to be a reasonable approach. The adoption of an additional factor of safety of 2 that is applied to the COV_2 for long-term strength in C737 (**Page 62**) is not required.

In this example, a partial factor for the material properties is 1.5 as calculated in the preceding pages of this guide.

SITE CLASSIFICATION PROFORMA

Product evaluation form

Proforma confirmation checklist information to be completed by geocellular manufacturer.

NB: pre design/construction Information held to be passed to geocellular manufacturer

Project title	BPF TOWERS
Manufacturer /supplier	MR PLASTIC MANUFACTURING COMPANY LTD.

Product type	WATERBOX 1
Product trade name	WATERBOX 1
Reference no.	PRODUCT CODE PMC WB1-01
Intended end use	ATTENUATION TANK.

Testing and confirmation checklist

Item		Item		Item	
Independent test certificates available	X	Quick compressive strength tests undertaken (min 8 tests)	X	Creep-rupture (up to __ days @ 23 ± 2ºC)	X
Horizontal/lateral tests undertaken and reported	X	Inclusion in the design strength for material factor (see Table 5.2 in CIRIA C737)	X	Project specific design calculations undertaken	X
Product warranty available	X	Current manufacturer/supplier professional indemnity insurance (P.I.) in place	NO	Confirmation that no outstanding information is outstanding from third parties (eg overall scheme designer, geotechnical information)	X
Deflection of units after constructed installation provided/predicted	X	Access and maintenance considered, ie inspection method, fully accessible	X	Plastic blend/specification: source provided, eg recycled, virgin, supplier quality control plan	X

Void ratio, (internal/ external porosity) %	95	**Type of test**, eg 300 mm plate, full platen creep tests (length hours), creep rupture tests (length hours), other specialist tests (see Chapter 4 in CIRIA C737)	STANDARD BBA TESTS.

Unit strength*

Ultimate strength: (mean value)	Vertical	440	kN/m²	Horizontal (mean)	97	kN/m²	
Characteristic strength: 50 years	Vertical	124	kN/m²	Horizontal 50 years	27	kN/m²	Design life = 50 years
Design strength: (50 years)	Vertical	83	kN/m²	Horizontal (50 years)	18	kN/m²	Partial factor = 1.5 / 1.5
Reference strength:	Vertical	85.5	kN/m²	Horizontal	18.5	kN/m²	Design life = 50 years (FOS = 1.5 min value)

*Note : Value given is required to be defined against the definitions below (see Section 4.2.3, CIRIA C737)

Ultimate strength is the maximum recorded strength (assuming a 'peak' is identified) or the resistance recorded at a strain amplitude of 6%, whichever occurs first, in a quick compression test. This is not a design strength.

Characteristic strength of a unit is the strength at the given design life derived from creep rupture tests. This is a cautious estimate of strength from creep rupture tests minus two standard deviations. If specialist tests are carried out, then the cautious estimate of strength may be modified as outlined in Section 4.2.2, CIRIA C737.

Design strength is the characteristic strength modified by the appropriate material partial factor.

Reference strength is the characteristic strength at a design life of 20 years and should be used as a basis to compare different geocellular units.

Further comments/information SEE MANUFACTURER'S DECLARED VALUES + DERIVATION FOR FURTHER INFORMATION.

Product design compliant [X] Yes [ ] No

Name MR BOX Signature M Box Date 19/1/2017

Notes:

1. The design compliance refers to confirmation that the product is suitable and appropriate for installation in the environment / conditions and design life specified.
2. The proforma should be supplemented with manufacturer product specific information, and testing data.

CIRIA, C737 151

9.5 Step 4.4: Additional data to be appended to Product Evaluation Form

In addition to the data on the *Product Evaluation Form*, in this example the yield strength of the units is required. For this example, it is assumed that there is no yield test data and the yield strength is taken as 70% of the short-term ultimate strength. This is a characteristic value of short-term ultimate strength, derived using the mean strength and standard deviation, as described on **Page 62 C737**.

The evidence from cyclic loading tests on various polypropylene geocellular tanks indicates that fatigue from low level and relatively infrequent cyclic loads does not cause premature failure (for example, from daily traffic by a few HGVs). This is because polypropylene is resistant to fatigue failure and testing on units has shown that it is not an issue at stresses up to 40% of the short-term strength and application of 21,000 load cycles. There is currently no standard method of cyclic or fatigue load testing. If units are to be subject to very frequent cyclic loading, for example, under a road designed to carry tens or hundreds of HGVs per day or in a rail environment (i.e., a Traffic Zone E2 and E3 - Class 3 design), then cyclic load tests that replicate the service conditions should be carried out.

For some units, testing with a 300 mm diameter plate gives a lower strength than with a full plate. If the strength parameters are derived from full plate tests, the supplier should confirm that these give the lowest strength.

The deflection of the units under short-term loads (elastic deflection) and during creep should be stated.

The creep coefficient is defined on **Page 169 of C737** and is taken from the straight-line portion of the deflection vs time graph for an appropriate test load that slightly exceeds the design load. In Table 2, the vertical creep coefficient is provided by the unit supplier for four different test loads that cover an equivalent soil cover depth from 1 m to 2.5 m (assuming a soil unit weight of 20 kN/m^3). The lateral creep coefficient is also provided at four test loads that cover an equivalent depth from 1.5 m to 3 m (assuming an active earth pressure coefficient of 0.33 and soil unit weight of 20 kN/m^3).

Table 2 Data to be appended to the Product Evaluation Form

Data to be supplied by manufacturer	**Details supplied**	
Manufacturer	Mr Plastic Manufacturing Company Limited	
Unit reference	WaterBox 1	
Test house	Box Squashing Inc	
Date of tests	3 April 2014	
Number of units tested	50 for all tests	
Confirmation that full plate tests give greater strength	Yes	
	Declared values	
	Vertical	**Lateral**
Mean of short-term compression results (10 minute tests)	440 kN/m^2	97 kN/m^2
Characteristic long-term creep rupture strength at 50 years	124 kN/m^2	27 kN/m^2
Characteristic short-term strength (at yield)	290 kN/m^2	64 kN/m^2
Short-term elastic deflection (load in kN to cause 1mm of deflection in the tank)	76	35
Creep coefficient for 50-year design life	0.49 at a load of 20 kN/m^2	0.63 at a load of 10 kN/m^2
Creep coefficient for 50-year design life	0.51 at a load of 30 kN/m^2	0.65 at a load of 13 kN/m^2
Creep coefficient for 50-year design life	0.54 at a load of 40 kN/m^2	0.68 at a load of 17 kN/m^2
Creep coefficient for 50-year design life	0.58 at a load of 50 kN/m^2	0.72 at a load of 20 kN/m^2

10. Step 5: Design calculations and analysis

10.1 Step 5.1: Compare design load to design strength

The purpose of this step is to compare the design load to the design strength to assess if the tank can support the loads over the design life with the chosen factors of safety.

The analysis follows the approach described in Appendix F of this guide.

The sum of the factored load effects should be less than or equal to the sum of the factored resistances. As more than one type of resistance is involved (short-term and long-term) an interaction formula is used. A similar approach is taken in structural design if both bending and axial compression are being considered in a beam.

$$\frac{Q_{dP}}{P_{dL}} + \frac{Q_{dT}}{P_{dS}} + \frac{Q_{dH}}{P_{dL}} \leq 1.0$$

where:

Q_{dP} = Design permanent load pressure = Q_{ckP} x γ_{LFP} x γ_{sf}

Q_{dT} = Design transient load pressure = Q_{ckT} x γ_{LFT} x γ_{sf}

Q_{dH} = Design hydrostatic pressure = Q_{ckH} x γ_{LFH} x γ_{sf}

Q_{ckP} , Q_{ckT} , Q_{ckH} = characteristic pressures for permanent, transient and hydrostatic loads.

γ_{LFP}, γ_{LFT}, γ_{LFH}, γ_{LFA}, γ_{sf} = Load factor (permanent), load factor (transient), load factor (hydrostatic), load factor (accidental) and site factor.

	Project: BPF Towers	Page: 13
	Description: Example design	
	Designer: BPF Pipes Group	Date: Feb 2017

Design analysis

Design loads and strengths taken from previous calculation sheets. Loads Section 8.6, Calculation sheet Page 9, Section 8.7, Calculation sheet Page 10 and Strengths Section 9.3, Calculation sheet Page 12.

Design equation

$$\frac{Design\ permanent\ load}{Design\ long\ term\ strength} + \frac{Design\ short\ term\ load}{Design\ short\ term\ strength} < 1.0$$

In this example, the hydrostatic load is assumed to be zero in the vertical direction and is included in the permanent load in the lateral direction.

Vertical

$$\frac{32.40}{82.70} + \frac{102.0}{193.3} = 0.92$$

Less than 1.0, so OK

Lateral

$$\frac{10.61}{18.0} + \frac{2.18}{42.7} = 0.64$$

Less than 1.0, so OK

	Checker: BPF Pipes Group	Date: 8/03/2017

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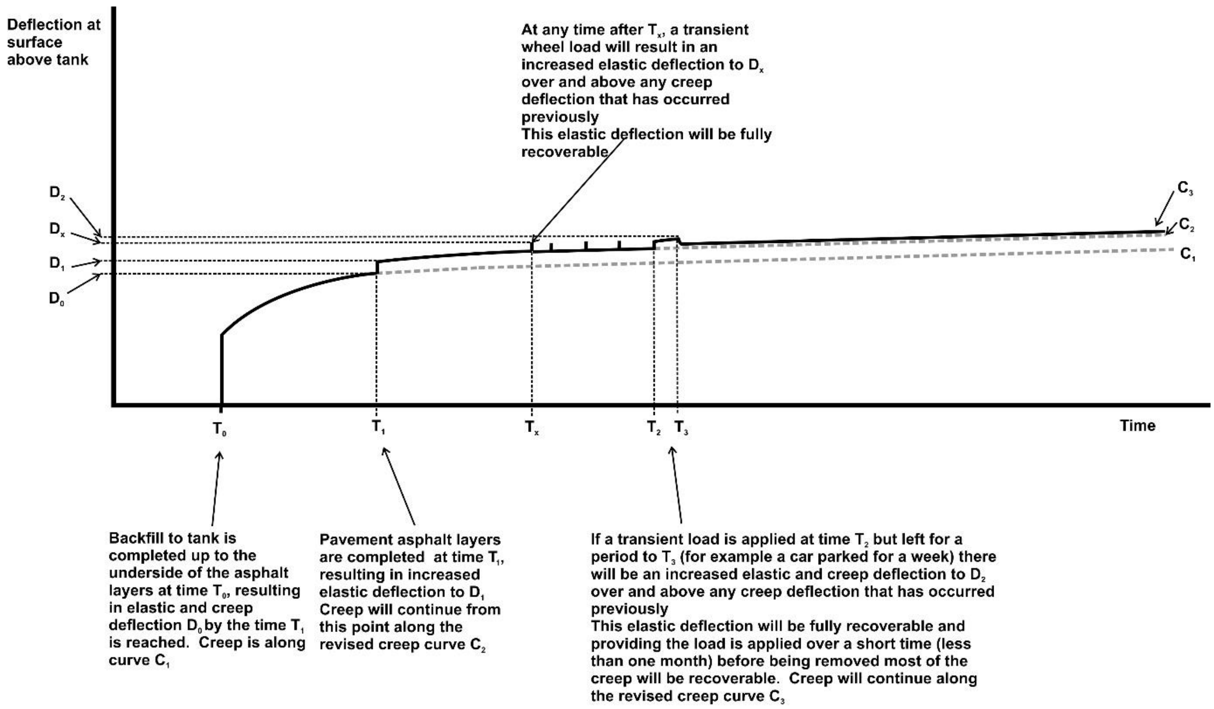
10.2 Step 5.2: Compare predicted tank deformation to acceptable limits for the site

The purpose of this step is to estimate the likely deflections under short-term and long-term loads.

Example calculations are also provided on **Pages 167 - 171 C737**.

The calculations in C737 suggest that elastic deformation under wheel loads is built out during construction. This is not the case because although the initial elastic part of a creep curve is built out during construction when the permanent backfill is placed, if an additional wheel load is applied to the backfilled tank, further elastic deflection will occur. This is normally fully recoverable on unloading. If transient loads are left for a period, some creep will occur but a significant proportion of this will be recoverable as well. This is shown in Figure 6. Experience has shown that if tanks are designed following this example, then the influence of cars being parked over the top of tanks for up to 8 hours per day should be negligible.

Figure 6 Deflection at ground surface during and after construction



This example assesses the influence of short-term wheel loads on deflection once the tank is installed.

The characteristic loads are the same as those derived earlier in these calculations.

Deflection is a serviceability limit state and appropriate partial factors of safety are used to determine the design loads. In this example, the load factors from **Table 5.9 of C737** are all equal to 1.0.

Creep over time under backfill

In this example, it is assumed that the short-term and long-term construction deflections will occur over a 12-month period as the tank is backfilled to the underside of the asphalt level after backfilling the tank and the asphalt pavement layers are constructed later towards the end of the project.

The creep coefficient taken from the supplier's data is 0.51 vertically (for a load of 30 kN/m^2 which is higher than the design permanent load of 24 kN/m^2). The creep coefficient is 0.63 laterally (which is for a load of 10 kN/m^2, higher than the design permanent load of 7.84 kN/m^2). In both cases, this will result in a slight over-estimation of the estimated deflection.

Allowable creep deformations of 10 mm laterally and 5 mm vertically should not cause problems to most road or car park surfaces. Greater allowable limits may be acceptable if agreed with the client and an assessment of the serviceability of the tank and overlying construction is made.

	Project: BPF Towers	**Page:** 14
	Description: Example design	
	Designer: BPF Pipes Group	**Date:** Feb 2017

Deflection

Characteristic loads from previous sheets:

Vertical variable = 68 kN/m^2 (overlap of wheel zones)
Vertical variable = 37 kN/m^2 (no overlap)

Vertical permanent load = 24 kN/m^2
Lateral permanent load = 11.2 kN/m^2

Partial factor of safety for serviceability limit state = 1.0 (for all load cases)

Therefore, Design loads = Characteristic loads x 1.0

Vertical creep deflection
Applied permanent load = 24 kN/m^2
Therefore, use creep coefficient from test at 30 kN/m^2 = 0.51
Creep at 12 months (8760 hours) = 0.51Ln(8760) = 4.6 mm

Creep at 50 years (438,000 hours) = 0.51Ln(438,000) = 6.6 mm

Creep after pavement construction = 6.6 – 4.6 = 2 mm (this is acceptable)

Lateral creep deflection
Applied permanent load = 11.2 kN/m^2
Therefore, use creep coefficient from test at 13 kN/m^2 = 0.65
Creep at 50 years (438,000 hours) = 0.65Ln(438,000) = 8.5 mm

This is less than 10mm and is acceptable

	Checker: BPF Pipes Group	Date: 8/03/2017

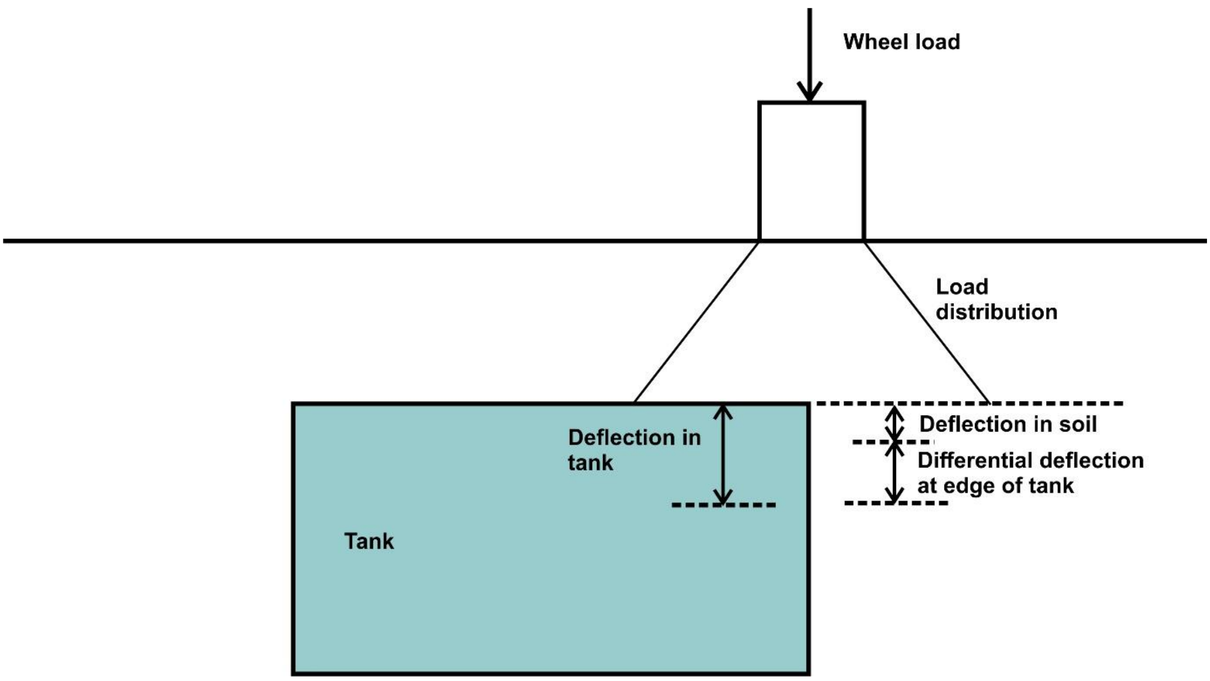
Short-term vertical deflection from wheels

This calculation follows the approach described on **Page 170 of C737**.

It is assumed that short-term vertical deflections due to the weight of backfill are built out.

The worst-case situation for tank deflection and its impact on the surfacing materials will be at the edge of the tank. Soil is normally a lot stiffer than the tanks and the differential movement will be at a maximum at this location (Figure 7). Differential movement is assumed to occur over the width of the wheel overlap.

Figure 7 Tank deflection relative to surrounding area



The allowable maximum vertical movement of a concrete block pavement surface under a wheel load is 1.5 mm in this example. This is a value that has been widely used for tanks that are covered by concrete block paving and that are only subject to occasional traffic by the maximum design load. Other limiting values could be used in agreement with the pavement design engineer and/or client. The more sensitive the surfacing material is to movement or the more frequent the deflection occurs the lower the allowable value is likely to be.

These examples ignore any deflection in the soil materials above and below the tank. Normally this is negligible compared to deflection of the tank. There is no need to consider this in routine designs. Therefore, the analysis uses a limiting deflection in the tank of 1.5 mm.

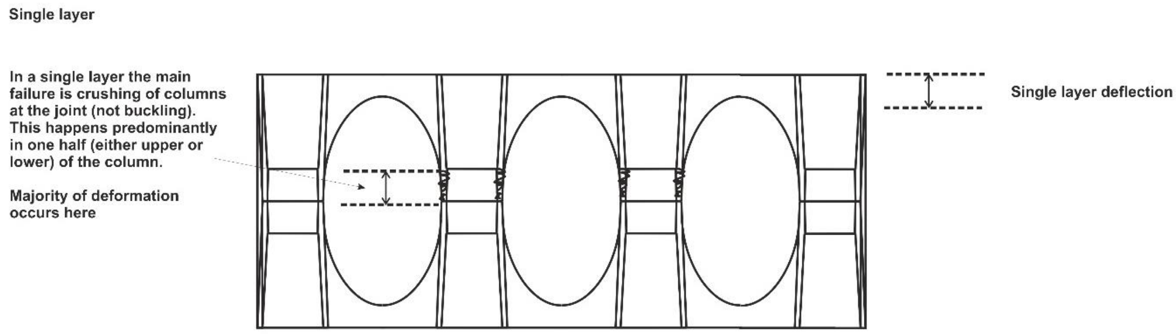
The elastic short-term deflection is taken from the supplier's data and, in this case, is 1 mm per 76 kN/m^2 applied load.

The allowable differential deflection (curvature) for a car park is 1 in 100 to 1 in 200 (**Page 170 C737**).

In this example, testing for the units has shown that under concentrated loads such as wheel loads, the deflection that occurs in the top of one layer of units does not increase if there is more than one layer. This is because the load is dissipated to a negligible level at the top of the second layer of units. Also, failure within the structure occurs by localised yielding of columns at a certain location, which is where most of the deflection occurs. This always occurs in the top layer of the example units and there is very little deflection in the lower layers (Figures 8a and 8b).

Alternatively, if the deflection that occurs in the unit is distributed evenly throughout the structure then the design deflection can be calculated using the strain for the units and the total height of the tank. Advice on the most suitable approach will be provided by the supplier.

Figure 8a) Different deformation modes - Failure (and deflection) caused by localised yielding within unit structure



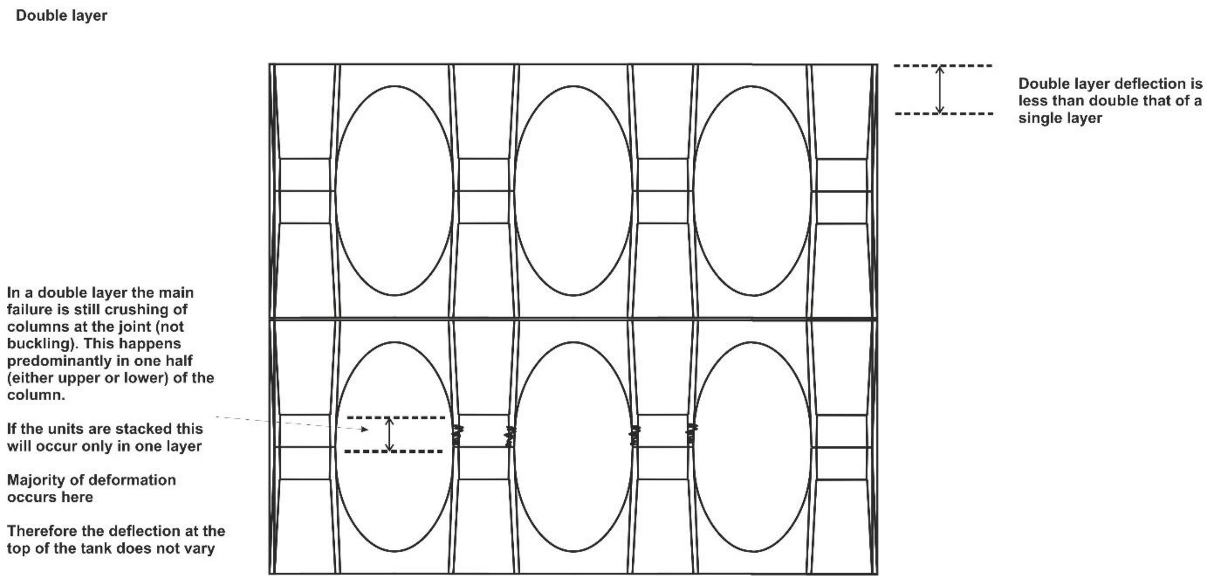
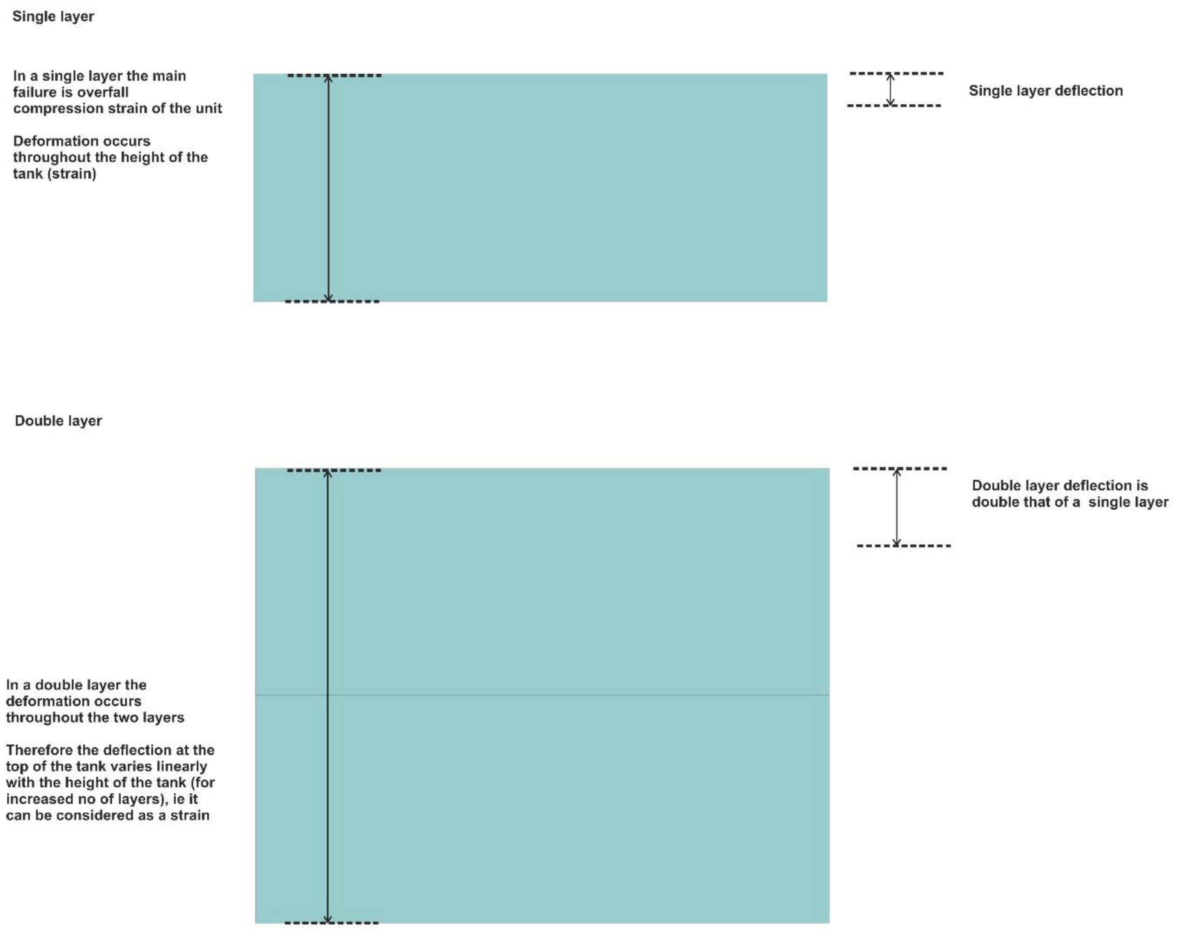


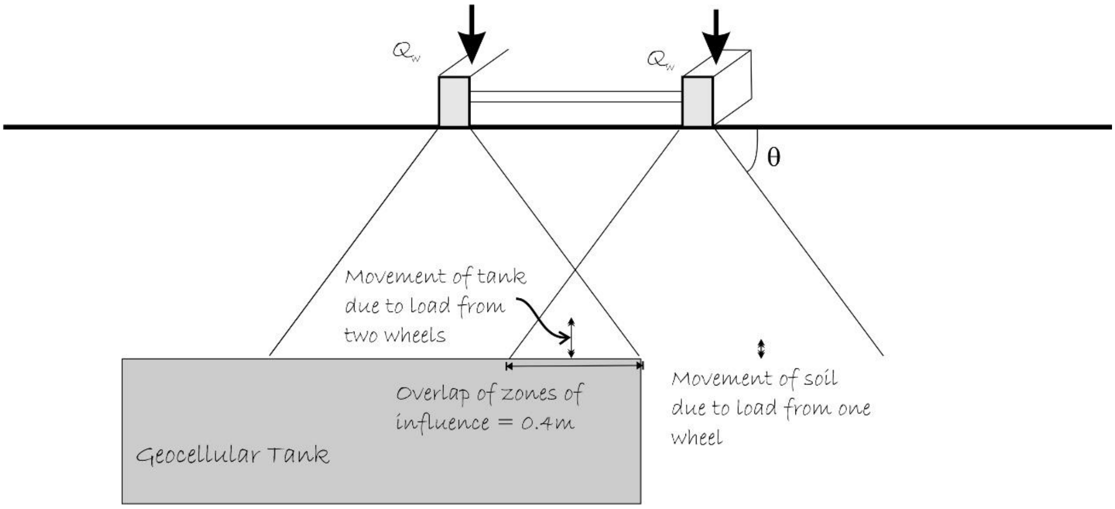
Figure 8b) Different deformation modes - failure (and deflection) caused by overall strain in a unit



Vertical short-term deflection

From product data deflection = 1 mm for every 76 kN / m² of applied load.

The worst case differential deflection will be where the overlap of the zone from the wheels is close to the edge of the tank.



Deflection outside overlap → 1 mm per 76 kN / m²

Load = 37 kN / m²
Deflection = 37 / 76 = 0.5 mm

Deflection inside overlap

Load = 68 kN / m²
Deflection = 68 / 76 = 0.9 mm (less than 1.5 mm OL for max deflection)

Differential movement = $\frac{0.9-0.5}{400}$ = 1 in 1000 (less than 1 in 200, so OK)

Checker: BPF Pipes Group | Date: 8/03/2017

10.3 Step 5a: Global deformation and site stability assessment
(Pages 100 and 101 C737)

The purpose of this step is to assess any global (or overall) stability issues as described on **Pages 100 and 101 C737**.

Global deformation and site stability checks are not required routinely and are only completed if there are site-specific concerns.

Interaction checks such as assessing nearby slopes, building foundations, etc. (**5.3.5.2 C737**) are not required routinely and are only completed if there are site-specific concerns.

In this example, where the site is flat and the tanks are outside the zone of influence of any structures, there is no requirement for a global stability check. If global stability is to be checked, it is likely to require input from a specialist geotechnical engineer.

Assessment of uplift or flotation (P**age 95 C737**) is not included in this example because the tank is above the water table. Uplift does need to be considered if the tank is likely to be below the groundwater at any time (seasonal variation in levels need to be considered). It is also required where a tank is constructed in clay soils and the water level in the backfilled excavation could rise over time. In online tanks, any water that does infiltrate the backfill can usually seep away along bedding and surround of outlet pipes.

11. Step 6: Prepare geotechnical design report

(Page 114 C737)

The purpose of the geotechnical design report is to summarise the critical assumptions and parameters used in the design calculations. This is a requirement of Eurocode 7.

The purpose of the report is to make those building the tank aware of the critical design factors and assumptions made. The most effective form of communication is a short one- or two-page summary of the information (including a diagrammatic ground conceptual model).

Any communication of relevant unusual risk that is required under the CDM Regulations should also be included here as well as on the design drawings.

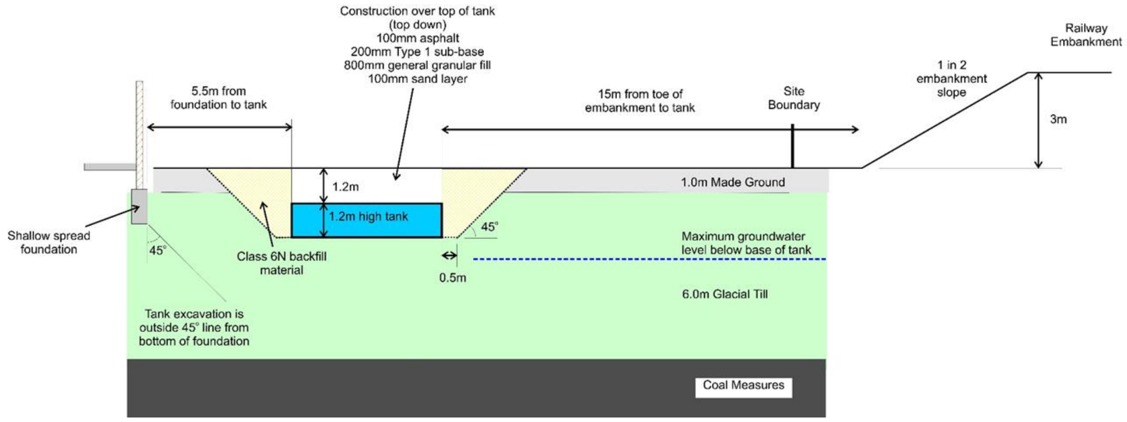
The geotechnical design report for this worked example is provided on the following two pages.

Table 3 Example geotechnical design report

Project Title: BPF Towers	**Job No:** DC01023
Tank Reference: Attenuation 1	**Made by:** SAW
Site Classification: 1	**Checked by:** SM
Relevant Reports with Factual and Interpretative Information: Dr Dirt Limited, Site investigation report for BPF Towers. V2 September 2016	**Critical Assumptions in Calculations Regarding Ground Conditions** Stratigraphy – see CGM below Parameters – See CGM below Excavation batter at 45˚ 0.5 m working space around tank at base (flat area) 100 mm of sand over top of tank General granular fill over top of tank Class 6N around sides compacted using light plate compactor (maximum force per blow 15 kN)
Relevant Codes and Standards: Eurocode EC 7 BS 5930	**Type of Units and Manufacture** Mr Plastic Manufacturing Company Limited WaterBox 1 Refer to manufacturer's installation guidelines
Description of Relevant Aspects of Site and Surroundings: Tank is located outside zone of influence of buildings or embankments It is assumed that the site is level.	**Information to be Verified During Construction** Ground profile in side of excavation Glacial till is present as firm to stiff clay in base of excavation Φ value of Class 6N Excavation batter is at 45° with 0.5 m working space Verification of geomembrane wrap to tank in accordance with CIRIA C735
Critical Assumptions in Calculations Regarding Loads Load Class = C Type of vehicles = cars and accidental passage of HGVs Construction traffic = construction traffic only allowed after completion of pavement to final level. Tank must be fenced off prior to this No cranes or similar	**Maintenance and Monitoring Requirements** No regular requirements for structural/geotechnical purposes See drainage design for specific requirements relating to hydraulic performance and silt removal

Table 3 Example geotechnical design report (continued)

Conceptual ground model (CGM) assumed in the design



Ground properties – those highlighted in blue to be checked on site during excavation and installation

Stratum	Typical thickness assumed in design	Unit weight assumed in design	Effective angle of friction assumed in design
Made Ground (medium dense black sandy GRAVEL of ash and clinker)	1.0 m	18 kN/m^3	32°
Glacial Till (firm to stiff dark grey silty sandy CLAY with much fine to coarse gravel)	6.0 m	20 kN/m^3	28°
Coal Measures (not investigated). Geological map indicates series of mudstone, siltstone, sandstone and coal seams. No workings.	100 m+	n/a	n/a
Class 6N backfill to Specification for Highway Works	--	18 kN/m^3	36°
Class 1 General granular fill to Specification for Highway Works	--	18 kN/m^3	32°

12. Additional information

12.1 Existing tanks

The BPF Pipes Group has published a position statement with respect to the use of C737 and C680 (https://bpfpipesgroup.com).

12.2 Testing

The short- and long-term compression tests on geocellular units are not like tests on small samples of materials (for example, tensile tests on strips of plastic material or compression tests on solid cubes of concrete). An individual geocellular unit is a complex structure, not a solid, and compression tests are used to obtain an indication of its performance in service. The use of the test results allows a simplified design method to be used. The alternative would be to carry out a complex structural analysis of the units for every site, which is prohibitively time consuming and not practical.

There are currently no published standard test methods available, although work is in progress to develop these. Most units available in the UK market have been tested following the methods described in C680 or very similar approaches that are relatively consistent between manufacturers. Experience has shown that providing the overall principles described in the methods described in C680 are followed, the practical impact on the quoted strengths is limited. It is important to remember that this is a practical engineering exercise and not a detailed scientific investigation. The levels of accuracy in the test method should reflect that. **Once European or UK Standard test methods are published these should be adopted for testing the units.**

Most units on the market have strength data that is based on tests completed using the basic approach described in C680. The short-term tests have been completed using a failure time of 10 minutes. Extending the failure time for short-term tests as suggested in C737 is not considered necessary. Tests on other plastic materials such as geogrids determine the short-term strength using much quicker loading rates.

Where suppliers have completed the more specialist tests described in C737, these results can be used. If such tests are not available, the strength parameters can be derived (conservatively) using the alternative approaches that are explained in this guide.

There are practical issues at present with completing some of the new tests listed in C737 as well as availability of laboratory time across Europe. One issue is determining the fatigue strength by cyclic load testing. Issues that need to be resolved before a standard test method can be published include determining an acceptable level of control over the loading cycles (the load varies quite significantly with each cycle unless very expensive control machinery and jacks are used) and the test duration.

Many suppliers have creep and creep rupture tests that have been completed using 300 mm diameter plates because of practical and safety issues when using full plates on units with a large surface area on the top or side faces. This data is valid if the correlation between the short-term strength derived from full plate and that from 300 mm plates is known. The correlation can be used to adjust 300 mm plate creep results to full plate values. Experience has shown that this is a valid approach and that the variability shown in short-term tests with various plates is reflected in creep and creep rupture tests.

Long-term creep test duration depends on the required design life. It can be a minimum of 2,000 hours (**Page 60 C737**). The longer the test duration the lower the partial factor and the higher the design life. The creep time is divided by 100 to give the design life (in years) as follows:

10,000 hours required for a design life of 100 years.

5,000 hours required for a design life of 50 years.

2,000 hours required for a design life of 20 years.

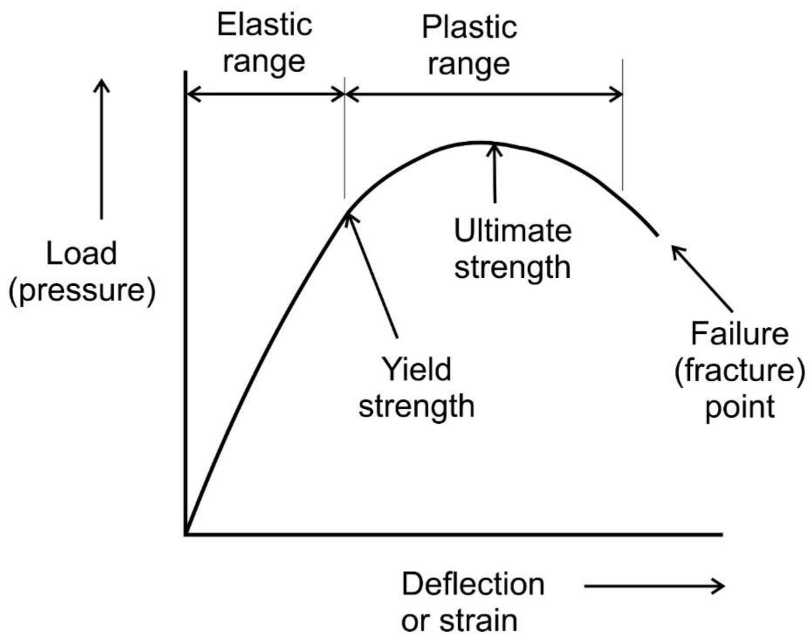
If a full suite of creep and creep rupture test data is not available, the creep strength can be determined using reduction factors applied to the short-term strength. This approach is used in Australian Standard AS 4678–2002, *Earth-Retaining Structures*. Creep reduction factors for different materials are given in that standard (Table K3). For polypropylene, the reduction factors are 0.2 for a 30-year design life and 0.17 for a 100-year design life.

The yield strength can be defined using the test approach in C737 with loading and unloading (a standard test method needs to be developed) or the following alternatives can be adopted:

- Assume that the yield strength is 70% of the peak failure strength from short-term tests (assumed in this worked example).
- Using the short-term test data, consider the intersection point of the elastic zone from the stress/deflection curve with that of the plastic zone (see Appendix G of this guide). This is the approach adopted in many current BBA certificates.

The yield strength is the point after which the material begins to deform plastically whereas the ultimate or peak strength is the maximum load that the unit can withstand. This is shown in Figure 9.

Figure 9 Determination of yield strength from short-term compression tests



Appendix A: Summary of key features of C680, C737 and the BPF Pipes Group guide to C737

	C680	C737	BPF Pipes Group guide to C737
Vertical loads	Uses DIN 1072 for traffic loads Has been shown to give reasonable designs since first use in 2001	Uses EC7 to define heaviest loads for bridge design Adopts loads from Lane 1 Defines accidental loads Results in higher characteristic concentrated loads than C680	Uses same approach as C737, based on EC7 Adopts loads from Lane 1, 2 or 3 depending on use of site Defines accidental loads Results in similar characteristic concentrated loads as C680
Lateral loads	Limited to designs up to depths of 4 m Assumes active earth pressure is mobilised Uses simple surcharge to analyse live loads	Allows design for depths greater than 4 m Assumes active earth pressure is mobilised for tanks up to 3 m deep Earth pressure is between active and at rest value for depths between 3 m and 4 m At rest earth pressure for depths greater than 4 m Requires specific analysis of concentrated load to side of tank and braking forces perpendicular to tank in accordance with EC7 for bridge design	Allows design for depths greater than 4 m Assumes active earth pressure is mobilised for tanks up to 3 m deep Earth pressure is between active and at rest value for depths between 3 m and 4 m At rest earth pressure for depths greater than 4 m Explains how to analyse concentrated load to side of tank. Does **not** require specific analysis of braking forces perpendicular to tank in accordance with EC7 for bridge design
Partial load factors	Standard structural and geotechnical load factors Dynamic factors optional, depending on speed and amount of turning and braking expected No site importance factor	Standard structural and geotechnical load factors Double counts dynamic factors for LM1 (which has dynamic factor included) Introduces additional site importance factor	Uses similar load factors to C737 but removes double counting of dynamic factors Explains what values are appropriate for each factor Explains what values are reasonable to use for site importance factor
Characteristic strength of the geocellular units	Relied mainly on short-term strength at yield Recommended vertical creep tests at a range of loads (in effect creep rupture tests) No requirement for lateral creep tests	Requires design to be based on characteristic strength obtained from creep tests, including creep rupture tests (i.e., a long-term value) Yield strength determined from special tests Specifically states that short-term tests are not to be used in design	Requires consideration of both short-term and long-term characteristic strength of units. Short-term tests are used to derive a short-term characteristic strength and creep rupture and creep tests are used to derive a long-term characteristic strength Uses short-term tests as part of design. In absence of specific tests allows yield strength determined from short-term tests using proportion of peak failure strength
Long-term creep	Recommended creep tests at various loads (which would by default include creep rupture tests) in vertical direction only. Minimum 5000 hours Limited long-term permanent vertical load to less than 20% of short-term strength (no FOS applied in this assessment) No requirement for specific assessment of creep in lateral loading (although the overall approach did include an allowance to limit creep in the lateral direction)	Creep rupture and creep tests required to define both vertical and lateral characteristic strength. Minimum 10,000 hours Implies that transient loads (traffic loads) should be assessed against long-term strength. FOS applied in all creep assessment	Creep rupture and creep tests required to define both vertical and lateral characteristic long-term strength. Minimum 10, 000 hours Long-term strength only used to assess permanent loads Short-term characteristic yield strength used to assess short-term transient loads (i.e., traffic) FOS applied in creep assessment Removes arbitrary doubling of coefficient of variation (COV) that is suggested in C737

	C680	C737	BPF Pipes Group guide to C737
Partial material factors	Short-term – 2.75 Long-term – 1.0 to 1.5	Long-term – varies depending on several factors from 1.5 to 2.7 for permanent works. Can be excessive if applied without thought Short-term – N/A	Long-term – varies depending on several factors from 1.5 to 2.7 but gives advice on reasonable values Short-term – varies depending on several factors from 1.5 to 2.7 but gives advice on reasonable values
Analysis	Vertical and lateral short-term and long-term loads considered against factored short-term strengths Long-term vertical loads compared to long-term strength Analysis of elastic settlement under traffic loads	Short and long term vertical and lateral loads compared to long term strength Overly theoretical discussion of settlement for buildings and no clear advice on assessment for geocellular tanks	Long-term vertical and lateral loads compared to long-term strength Short-term vertical and lateral loads compared to yield strength Interaction formula used to balance short- and long-term assessment Simplified assessment of short-term and long-term settlement with clear advice on analysis of geocellular tanks (based on approach that has been proven to work in practice for most types of tank)

Appendix B: Traffic zones and site classification

The first step in the design process is to classify the site and proposed tank construction. The classification then guides the designer to the appropriate test and checking requirements for the application. This classification system is consistent with the categories used in the Design Manual for Roads and Bridges (Highways England, 2012)[2] which states that the boundaries of each Category from 0 to 3 are not rigid and each proposal would be decided on its own merits.

The principle aim of the *Design and Construction Classification* system in **C737 (Chapter 3)** is to identify geocellular tank installations that have high intrinsic complexity or where the consequences of failure are severe. Severe consequences could be in terms of the health and safety considerations associated with a sudden collapse or the economic cost and disruption/congestion to traffic caused by a more progressive failure. Experience indicates that sudden catastrophic collapse of geocellular structures is not likely to occur and if collapse does occur it would be a slow progressive mechanism.

More complex or high-risk situations require more comprehensive testing to support the design and detailed checks by qualified professionals. However, it is likely that most situations will fall within Category 1 or 2, for which routine testing and design checking will be sufficient.

Traffic Zones

Different traffic zones may be based on consideration of:

- The influence of the tank on the road, car park or hardstanding pavement structure; or
- The traffic loads that will be applied to the tank.

The following zones have been identified for application to geocellular tank design.

Based on influence of tank on road pavement or structures (or vice versa)

1	Close to foundations or retaining walls– defined in C737 as within h + 2 m (see **Figure 3.1 C737**).
2	Close to slopes or stockpiles - defined in C737 as within h + 10 m (see Table B1). However, this is considered to be conservative and site-specific slope stability analysis may allow the 10 m distance to be reduced. For slope heights less than 2 m and tank depths less than 3 m, the distance can be reduced to h + 5 m without further analysis.
3	Any part of the tank is within a 45° line of influence from underside of carriageway construction.
4	Outside the zone of influence from any structures, slopes, stockpiles or road pavement.

[2] Design Manual for Roads and Bridges, Volume 1, Section 1, Part 1, BD2/12, Technical Approval of Highway Structures. 2012.

Based on traffic load

A	Anywhere that vehicle access is not possible (e.g., due to fences or barriers, road layout or topography).
B	Anywhere that only cars can access due to physical constraints.
C	Anywhere that HGVs will only access as an "accidental load" (i.e., not regular HGV traffic, for example, vehicle overrun on a verge at the back of a footway).
D	Anywhere that is subject to limited HGV traffic at very low speed (<15 mph) such as fire tenders and refuse trucks.
E	Everywhere else (assumed to be subject to regular unrestricted HGV traffic). This category is split into three sub-categories depending on the type of HGV loading that is expected (E1 to E3). E1 is for areas where HGVs will be regular and moving at low speeds such as lorry parks and loading bays. E2 would cover some estate roads in residential developments and E3 would cover trunk roads and motorways. In the latter case, in the running lanes of motorways (including the occasional hard shoulder on Smart Motorways), specific assessment of the special vehicle loads should be undertaken to the requirements of Highways England.

The zone for a tank will be a combination of the position in relation to zones of influence (1 to 4) and the likely traffic load (A to E). For example, a tank that is outside the zone of influence of any structures or roads and is not accessible to vehicles would be defined as Zone 4A.

Examples of traffic zones

Examples of situations that are typical of each of the traffic zones A to E are shown in Table B.1. There is no consideration of the zone of influence in the Table (i.e., all situations are considered to be Zone 4).

Table B.1 Example traffic zones

Zone	Description	Examples
A	Anywhere that vehicle access is not possible (e.g., due to fences or barriers).	Triangular area between Motorway and slip road is not accessible to vehicles.
		Wide central reservation on approach to a bridge is not accessible to vehicles due to barriers.
		Grassed area in a roundabout under flyover is not accessible to vehicles due to bank and shrub/bush vegetation.
B	Anywhere that only cars can access due to physical constraints, e.g., width or height barriers.	Car park with a height restriction barrier.
C	Anywhere that HGVs will only access as an "accidental load" (i.e., not regular HGV traffic, for example, vehicle overrun on a verge at the back of a footway).	A wide verge behind a footway
		The grassed area of a roundabout is not readily accessible to HGV traffic due to earth mounds.
D	Anywhere that is subject to limited HGV traffic at low speed such as roads with access for fire tenders and refuse trucks.	An example for a minor access road in a residential development is given in *Kent Design Guide. Section 2 Creating the Design. Step 3 Designing for Movement*[3].
E	Everywhere else (assumed to be subject to regular unrestricted HGV traffic).	An example for a local distributor road in a residential development is given in *Kent Design Guide. Section 2 Creating the Design. Step 3 Designing for Movement*.

[3] https://www.kent.gov.uk/__data/assets/pdf_file/0018/12096/design-guide-movement.pdf

Site classification for the traffic zones

Each of the preceding zones has been classified in accordance with the site classification system described in C737. Some adaptations have been made based on experience of using the system. The need to adapt the system is recognised in C737 which states:

"*The system will require further testing in use to allow modifications and developments to be made, as it is inevitable that not all circumstances will have been foreseen and a process of evolution is likely*".

A summary of the classification of the different traffic zones using the C737 methodology, together with the required design checks and testing, is provided in Table B.2. This is limited to locations outside the zone of influence to structures, slopes or road pavements.

Table B.2 Classification, design checks and testing requirements – based on traffic zones (outside any zone of influence to structures, etc.)

Traffic zone	General description	Type of site	Score	Use	Score	Information	Score	Topography	Score	Location	Score	Depth to base	Score	Cover (see note at base of table)	Score	Construction phase	Score	Classification: Total score	Classification: Class	Testing requirements	Recommended actions/roles (Table 3.2 C737)	Design requirements (Table 3.3 C737)	Checking requirements (Table 3.2 C737)
A	No vehicular access	Commercial	10	Attenuation	5	Assume all relevant information is available	0	Level ground	0	Equivalent to parkland	0	1 m to 3 m	5	0.3 m to 2 m landscaped	10	Assume some construction plant passing over	20	50	1	Long-term creep rupture and short-term tests (300 mm diameter and full plate)	Simple design calculations by competent building professional with relevant industry experience	Check units have sufficient strength to support vertical loads (distributed and concentrated). Check cover to units is sufficient to distribute concentrated loads and to prevent flotation. Assess earth and water pressure on sides using standard methods and assuming active earth pressure coefficients apply	Simple design checks to be undertaken by competent building professional. Independent check by another engineer who may be from the same team (Incorporated or Chartered Engineer to oversee checks)
B	Car access only	Commercial	10	Attenuation	5		0	Level ground	0	Equivalent to car park light use	15	1 m to 3 m	5	1 m to 2 m trafficked	15		20	70	1				
C	Accidental HGV access	Commercial	10	Attenuation	5		0	Level ground	0	Equivalent to car park general	20	1 m to 3 m	5	1 m to 2 m trafficked	15		20	75	1				
D	Limited HGV traffic at low speed	Commercial	10	Attenuation	5		0	Level ground	0	Low speed roads	30	1 m to 3 m	5	1 m to 2 m trafficked	15		20	85	2	Long-term creep rupture and short-term tests (300 mm diameter and full plate)	Design by Chartered Civil Engineer with 5 years 'post chartered' specialist experience in ground engineering	Check units as above. Consider allowable movements and assessment of manufacturer's data. Consider creep deformation. Detailed assessment of construction activities.	Design overseen by Chartered Civil Engineer with 5 years 'post chartered' specialist experience. Category 2 check by an Engineer who must be independent of the design team but can be from the same organisation
E1	Regular HGV traffic at low speeds	Commercial	10	Attenuation	5		0	Level ground	0	HGV park	30	1 m to 3 m	5	1 m to 2 m trafficked	15		20	85	2				
E2 and E3	All other locations. High speed HGV traffic	Commercial	10	Attenuation	5		0	Level ground	0	Equivalent to full highway loading	80	1 m to 3 m	5	1 m to 2 m trafficked	15		20	135	3	Long-term and short-term tests as above plus cyclic loading tests (fatigue test). Full-scale pavement tests if less than 1 m cover to tank	Design by Chartered Civil Engineer with Geotechnical Advisor status	As above plus assessment of fatigue and cyclic loading and detailed assessment of deformations. Numerical modelling required	Senior Specialist Geotechnical Engineer with Geotechnical Advisor status should be appointed to oversee design process, likely complex modelling and testing required. Category 3 check by an Engineer from a separate organisation to that of the designer.
NOTES:		Assume all locations are "commercial"		Assume attenuation is worst case. Note - there is no reason why attenuation is greater risk than soakaway so score for soakaway has been used				Assume for this first stage, level ground and outside zone of influence of walls, etc.						Assume >1 m but less than 2 m = 0. Not explicitly stated		Assume the tank is not below groundwater table		Assume tank is outside zone of influence of any structure etc. i.e. Zone 4		Assumes units are not prone to excessive bending or instability when subject to shear loads or other uneven loading (units assembled on site from plates require specific shear testing)			

Appendix C: Wheel and surcharge loads plus factors to be used to calculate characteristic traffic loads

Loads

Characteristic loads are a best estimate of the load likely to be placed on a structure during its design life. Factors of safety are applied to the characteristic loads derived for the permanent and temporary works. This is done in accordance with the following equation:

Design loads, $P_d = \sum(P_{ck} \text{ x } \gamma_{LF} \text{ x } \gamma_{df} \text{ x } \gamma_{sf})$

Where

P_{ck} = characteristic loads, γ_{LF} = load factor, γ_{df} = dynamic factor, γ_{sf} = site factor.

C737 states that it provides characteristic loads for guidance (**Section 5.3.4.1 C737**) and that the actual load to be considered for a specific site is a matter for professional judgement and requires careful consideration of the vehicles that could access an area. The following assessment provides justification for the loads that can be used to design tanks in the different zones within the highway boundary.

The characteristic loads proposed in **C737** (**Table 5.6**) are based on the design loads for bridges and other structures. They are taken from the Eurocodes for structural design and specifically the one related to loads on bridges (National Annex to BS EN 1991-2: 2003 *Traffic Loads on Bridges*). C737 also takes the worst-case values from the code, which are based on "international" HGVs. The Eurocode has several different load models (LM1 to LM3) to cover different types of traffic on bridges. The load models specify wheel loads and distributed loads to allow the efficient and safe prediction of bending moments and shear forces for the design of bridges. The loads in the models have been selected and calibrated so their effects represent the actual effects from traffic on bridges in European countries[4]. They are not actual wheel loads that occur in reality and, therefore, they may not be directly applicable to the design of geocellular tanks.

Load Model LM1 is intended to cover flowing, congested or traffic jam situations with a high percentage of lorries. It is based on 1000-year return period traffic on main roads in Europe. This return period is well above the design life of a geocellular tank, which currently is no greater than 50 years. The values used in C737 that are taken from LM1 are for the slow lane of a motorway where there is a high percentage of lorries. LM1 allows lower loads for Lanes 2 and 3 of a motorway (i.e., an increasing proportion of cars). For many geocellular designs in sites where there are only cars present or a small proportion of the traffic is HGVs, then the lower wheel loads for Lane 2 and 3 may be appropriate. The specified loads in Load Model LM1 include an allowance for dynamic effects and the dynamic impact factor should be 1 (**C737 Table 5.6** and **Table 5.10**) imply that a further dynamic factor should be applied, which is not correct).

Load Model LM1, BS EN 1991-2 allows a distributed load of 2.5 kN/m^2 in Lanes 2 and 3 of a motorway and only has 9 kN/m^2 in Lane 1. Pedestrian loads are also represented by a distributed load of 2.5 kN/m^2. The hard shoulder has no distributed load (although it would do on a Smart Motorway). It would, therefore, seem reasonable to allow a surcharge of 2.5 kN/m^2 in areas where there is no traffic loading. For areas where crowds may be present, a value of 5 kN/m^2 is used (the

[4] Veselin Slavchev (2012). Fast Calculation Model for EN 1991-2 Load Model 1 Using Equivalent Uniform Loads. Advanced Research in Scientific Areas, December 2012.

example used in BS EN 1991-2 is for a bridge leading to a sports stadium and is in Load Model LM4). Other areas with car traffic should use the UDL for Lanes 2 and 3 in LM1, combined with the value for α (2.2) to give a total UDL of 5.5 kN/m^2. For areas with regular HGV traffic (Zone E) use a UDL of 10 kN/m^2.

Load Model LM2 is for single axle loads and is not used in C737. The LM2 model is used in bridge design to simulate worst case forces in short span members such as deck slabs spanning between main beams. It is not relevant to the design of geocellular tanks.

Load Model LM3 is a set of nominal values that are based on special vehicles (SV) that fall outside the Road Vehicles (Construction and Use) Regulations 1986. SV vehicles comply with the Road Vehicles (Authorisation of Special Types) (General) Order 2003 or the Individual Vehicle Special Orders (i.e., vehicles commonly known as "abnormal loads"). C737 suggests this should be applied to the design of tanks below all public roads. This seems inappropriate (even with the adjustment factors included in C737) for many tanks below small estate roads or below verges where accidental loading may occur. Special vehicles are not likely to be present in many routine design situations. Therefore, the characteristic loads provided in **C737 (Table 5.6)** can be conservative and alternative values are proposed in Table C.1. These are based on Load Model LM1 except for situations where abnormal loads may be expected and Load Model LM3 is appropriate. Note that the LM1 Lane 1wheel loads are greater than all the special vehicle wheel loads so this will not be a concern unless special vehicles are likely to act as accidental loading in Zones A to C.

The recommendations in C737 also include the use of adjustment factors similar to those in BS EN 1991-2. These adjustment factors are included in BS EN 1991-2 to allow for differences in vehicle traffic between bridges due to its composition (e.g. percentage of lorries), its density (number of vehicles per year), conditions (e.g., likelihood of traffic jams and the likelihood of overloading). The adjustment factor is denoted as α when it is applied in Load Model LM1 and β when it is applied to Load Model LM3. Specific adjustment factors for the design of geocellular tanks in each Highway Zone are provided in Table C.1 where appropriate.

Load factors

Design loads are determined by multiplying the characteristic permanent and variable loads by the appropriate load and site importance factors (note that loads are called actions in Eurocodes). The dynamic factors are applied to variable loads (actions) generated, for example, by road or rail/metro traffic. Dynamic factors allow for increases in static forces due to braking of vehicles, etc. If traffic speeds are low (i.e., less than 15 mph), then dynamic factors would not normally be applied. Load Model LM1 already includes an allowance for dynamic effects and an additional factor is not required. Therefore, dynamic factors need only be applied if a design is considering Load Model LM3 or abnormal loads. Dynamic factors are applied for both ultimate and serviceability limit state checks and are outlined in **Table 23.5 C737**.

The intent of the site importance factor is to ensure the probability of failure is sufficiently remote, depending upon the site classification and associated consequences of failure. For all Zones except D and E, the site importance factor should be 1. For Zone D and E use 1.25 for ultimate limit state analysis.

Table C.1 Suggested loads and adjustment factors

					UDL	Wheel Load Normal Service Load Case							Wheel Load Accidental Load Case						
CIRIA traffic category (pervious surfaces)	CIRIA C737 description	Description for highways	Highways Zone	Typical vehicles/loading	Distributed (kN/m^2)	ULS wheel load	Details	Max wheel load in model (applied pressure at surface - kN/m^2) (half of axle load)	Dynamic amplification factor -DAF	Overload factor	Adjustment factor (α) to allow for lower loads than international HGV and 1000-year return period	Characteristic wheel load at ground surface (pressure - kN/m^2)	Accidental load Wheel load	Details	Max wheel load in model (applied pressure at surface - kN/m^2) (half of axle load)	Dynamic amplification factor -DAF	Overload factor	Adjustment factor (α) to allow for lower loads than international HGV and 1000-year return period	Characteristic wheel load at ground surface (pressure - kN/m^2)
0	Small domestic gardens (isolated from roads and vehicle access)	Anywhere that vehicle access is not possible (e.g., due to fences or barriers)	A	Pedestrian	2.5 (but increase to 5 if there is a risk of crowds)	None	-	0	-	-	-	0	Consider risk of trafficking where adjacent to drives or roads and possibly use LM1 Lane 3	100kN axle load on 400mm by 400mm contact area	313	1	1	1	313
1	Small domestic gardens (adjacent to drives or roads)	N/A																	
2/3	Car park (with height or width barriers to limit access)	Anywhere that only cars can access due to physical constraints e.g., width or height barriers	B	Car	5.5	LM1 Lane 3	100kN axle load on 400mm by 400mm contact area	313	1	1	1	313	LM1 Lane 3	100kN axle load on 400mm by 400mm contact area	313	1	1.5	1	470
4	Car parks without barriers	Anywhere that HGVs will only access as an "accidental load" (i.e., not regular such as vehicle overrun on a verge at the back of a footway)	C	Mainly cars with accidental HGV loading	5.5	LM1 Lane 2 α for "normal" HGV	200kN axle load on 400mm by 400mm contact area	625	1	1	0.8	500	LM1 Lane 2	200kN axle load on 400mm by 400mm contact area	625	1	1	1	625
5	Private roads or cul-de-sacs, access tracks (<15mph)	Anywhere that is subject to limited HGV traffic at very low speed such as fire tenders and refuse trucks	D	Cars and "normal" HGV at low speed	5.5	LM1 Lane 2	200kN axle load on 400mm by 400mm contact area	625	1	1	1	625	LM1 Lane 1	300kN axle load on 0.4m by 0.4m contact area (includes DAF)	938	1	1	0.8	750
6	HGV parks, loading bays	Everywhere else (assumed to be subject to regular unrestricted HGV traffic)	E1	Cars and "normal" HGV	10	LM1 Lane 1 α for "normal" HGV	300kN axle load on 0.4m by 0.4m contact area (includes DAF)	938	1	1	0.8	750	LM1 Lane 1	300kN axle load on 0.4m by 0.4m contact area (includes DAF)	938	1	1	1	938
6	Public roads, estate roads		E2	Cars and "international" HGV	10	LM1 Lane 1	300kN axle load on 0.4m by 0.4m contact area (includes DAF)	938	1	1	1	938	LM1 Lane 1	300kN axle load on 0.4m by 0.4m contact area (includes DAF)	938	1	1.2	1	1126
7+	Public highway (trunk roads)		E3	Cars and "international" HGV plus Special Vehicles	10	LM1 Lane 1	300kN axle load on 0.4m by 0.4m contact area (includes DAF)	938	1	1.2	1	1126	LM1 Lane 1	300kN axle load on 0.4m by 0.4m contact area (includes DAF)	938	1	1.5	1	1407

Note: Special vehicle loads give lower wheel loads than LM1 Lane 1

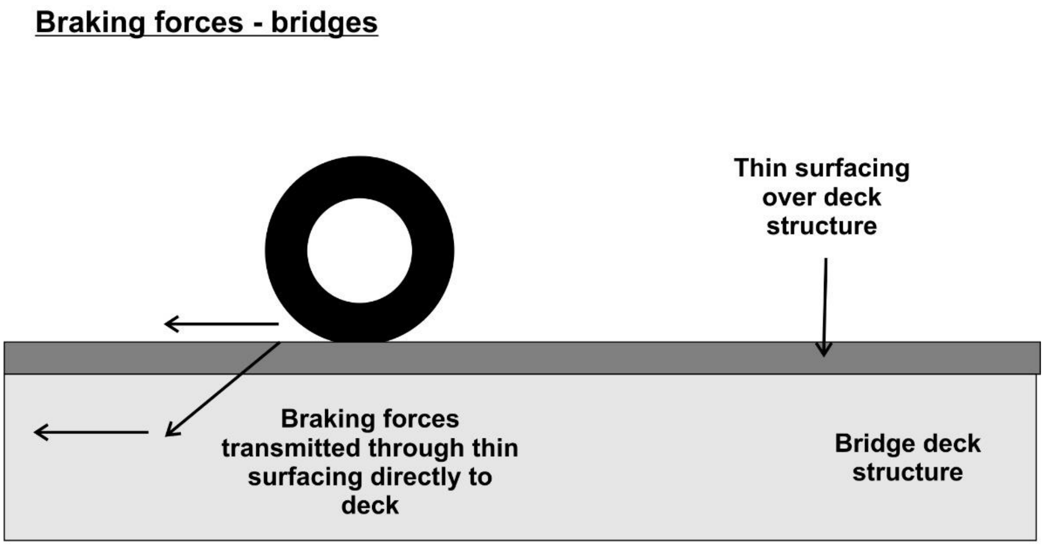
Appendix D: Braking forces

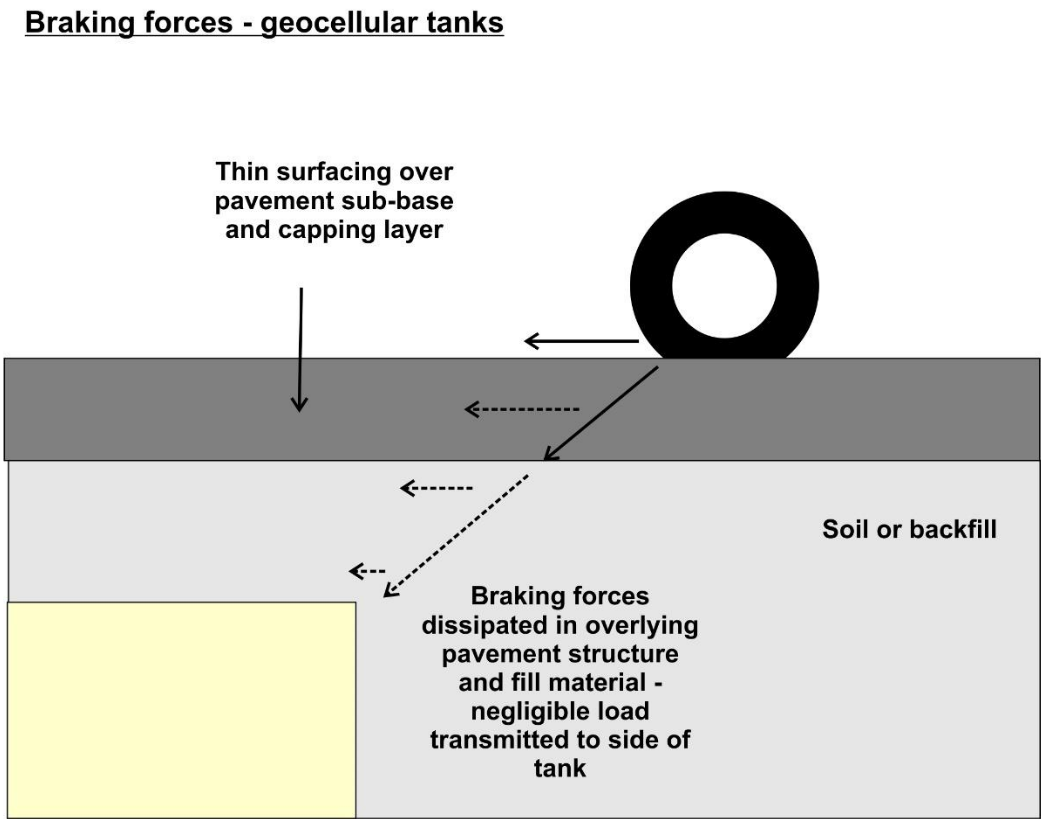
C737 (5.3.4.3) indicates that horizontal braking forces may be transmitted to tanks. For routine designs, the dynamic factor allowed for in the load models discussed above will be sufficient to allow for this where units have a cover of 0.6 m or greater up to load Zone C (mainly cars) and over 1.0 m in all other cases.

C737 suggests that braking forces on the side of tanks should be determined in accordance with EC7 assuming that the braking force is 60% of the vertical load. This approach from bridge engineering is over-conservative when applied to geocellular tanks that are at some depth below the application of the wheel loads which dissipate through the adjacent and overlying pavement structure and soils (Figure D.1 of this guidance). Horak et al[5] demonstrated that the horizontal shear forces from heavy aircraft braking and turning, such as a Boeing 747 "Jumbo Jet", would be dissipated to a negligible level within the top 100 mm of the pavement surface. Therefore, the preceding analysis, that takes account of the horizontal component of the wheel load located adjacent to a tank, is sufficient to allow for braking forces if vehicles drive onto a tank perpendicular to the edge. It is not considered necessary to carry out an additional analysis as suggested by C737.

[5] Horak E, Emery S, Maina J W and Walker B (2009). Mechanistic Modelling of Potential Interlayer Slip at Base Sub-base Level. Eighth International Conference on the Bearing Capacity of Roads, Railways, and Airfields, June 29 to July 2, 2009, University of Illinois, Urbana Champaign

Figure D.1 Comparison of braking forces on bridges and geocellular tanks





Appendix E: Lateral loads and arching

Introduction

Historically, the design of geocellular tanks using the guidance provided in C680 has not explicitly considered the effects of creep in the lateral design. There is some allowance for creep effects in the design using short-term lateral strength and a factor of safety of 2.75 (along with yield strength and tests in which failure takes at least 10 minutes). There is, however, no requirement in C680 for a specific lateral creep assessment as there is for vertical loading.

The reason for this is because it is understood that the geocellular tanks are flexible and, therefore, arching occurs in the soils to reduce the pressure on the side of the tank.

C737 has introduced a requirement to consider the lateral creep and it is apparent that using maximum earth pressures estimated using traditional earth pressure theory (as used for retaining walls) alongside long-term 50-year creep strength would significantly reduce the depths to which current modules can be installed.

Therefore, an assessment of the likely reduction in earth pressure on the side of tanks due to arching has been completed.

Evidence for arching effects

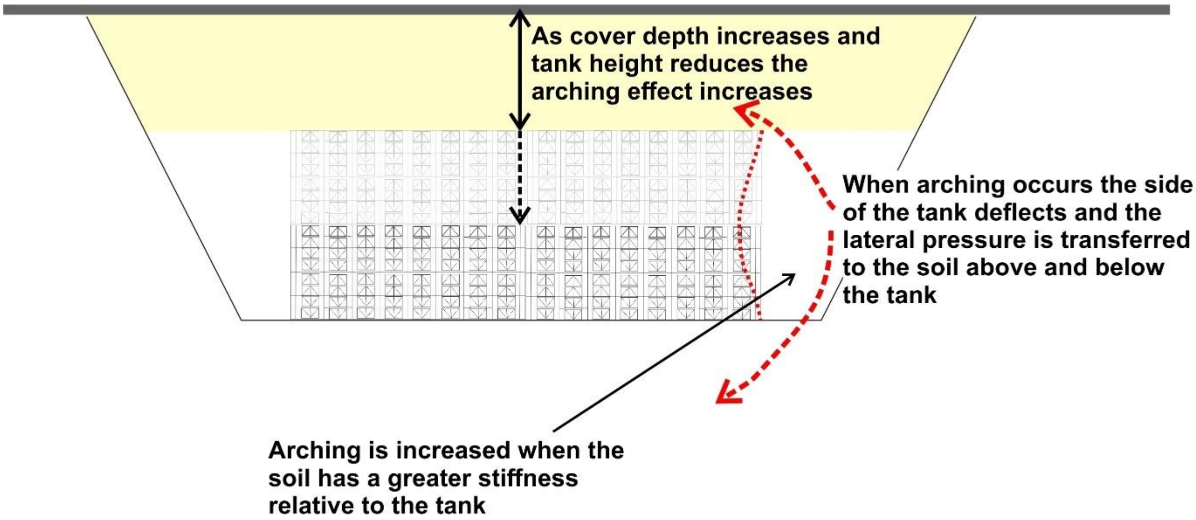
C737 identifies that arching can reduce the earth pressure applied to cells below the values predicted by simple earth pressure theory (see **Figure 2.39 C737**).

The graph shows the horizontal stress on the cell for different cell stiffnesses. For a tank:soil stiffness ratio of 0.5 MPa:8.6 MPa, i.e., 0.06, the maximum pressure in the tank is about 30% less than the pressure with no tank[6]. The text states that arching has reduced the pressure to values below the active pressure predicted by simple theory. As the stiffness of the tank increases, the earth pressure on the side increases until (for very stiff units where the stiffness exceeds the soil stiffness by a large amount) the earth pressure on the side of the tank is similar to the earth pressure with no tank.

The two main factors that will affect whether arching can occur are the ratio of cover depth to tank height and the ratio of tank lateral stiffness to soil stiffness (Figure E.1).

[6] Typical values of Young's Modulus for soil and granular material can be found in source document, Geotechdata.info, Soil Young's Modulus, http://www.geotechdata.info/parameter/soil-young's-modulus.html, updated September 2013.

Figure E.1 Arching around a geocellular tank



The effect of lateral yielding of earth-retaining structures and the reduction in earth pressure is also recognised in the design of reduced pressure retaining walls and bridge abutments. Polystyrene is used as a flexible layer at the rear of walls and it can reduce earth pressure to below active values[7].

Experience in the UK, Europe and Japan suggests that there is a significant difference between the theoretical earth pressure estimated on the side of geocellular drainage tanks and that which occurs in practice. The tanks are flexible buried structures and the soil structure interaction is complex and, therefore, the analysis is not as straightforward as it would first appear. There has been very little research in this area. However, BBA certificates in place for 10 years or so require manufacturers to advise of any failures and there have been no lateral failures reported. What is known is that the tanks have been used for over 20 years in Europe and Japan and there are very few reported lateral failures, despite suggestions that the lateral strengths are insufficient for the depths at which some are installed.

There are several possible reasons for the difference between simple theory and practice. The main reasons that earth pressure is overestimated are:

- Relaxation of structure and arching within the soil.
- Conservatism in design parameters.
- Cohesive soils take time to reach fully-drained conditions with c' = 0.

The usual method of estimating the earth pressure on the side of tanks is to adopt the approach from conventional retaining wall analysis. This assumes a rigid wall that is free to rotate. A plastic geocellular tank behaves differently to this and the reduced stiffness when compared to traditional materials means that the soil:structure behaviour is such that lateral pressures are unlikely to ever reach full active earth pressure. It also assumes that the only resistance to the applied forces is that provided by the geocellular tank. Again, this is not always the case, especially where the depth of cover soils is substantial. The cover soils can provide a significant amount of resistance against the wedge of soil that is mobilized during development of lateral active earth pressures. An analogy is the soil pressures on buried plastic pipes that are much lower than for rigid materials.

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[7] Koerner R M. Designing with Geosynthetics, 6th Edition, Volume 2, 2012

Creep under vertical and lateral loads occurs in plastic tanks. However, French design guidance in use since 1998 or earlier (Perrier, 1997[8]) has been to limit vertical loads to 10% of the short-term compressive strength of the individual units. There is no requirement to limit lateral loads. A similar approach is taken in Japan[9] but again there is no requirement to apply this limit to lateral loads. A study in France in 2007[10] inspected tanks that had been installed in various areas. A significant proportion of the tanks had been in place for over 10 years with no reported failures.

The simple method of design used at present assumes that the tanks behave in the same way as retaining wall structures. Numerical modelling of tanks has shown that the earth pressures experienced on the side of geocellular tanks are actually much lower than predicted by this simple analysis (C737). One potential reason for this is arching within the soil as described above.

Design parameters

Design parameters for tanks are usually based on soil descriptions. It is extremely rare that geotechnical tests are undertaken to determine the long-term shear strength parameters for use in estimating earth pressure. In most designs cohesion of soils is assumed to be zero.

This approach is invariably conservative with engineers assuming very low values for the angle of friction of made ground, for example, without considering if it is densely compacted or not. For naturally-occurring siliceous sands and gravels, the minimum angle of shearing resistance, ϕ, can conservatively be taken to be 30° [11]. However, through the effects of dilation, it can be up to about 17° higher depending on the angularity, grading and density of the material[12].

The time for clay soils to lose cohesion should also be taken into consideration, where appropriate. TRL Report 550 indicates that it can take a century to achieve equilibrium moisture conditions and loss of cohesion in clay soils. (Simple calculations show that a soil exhibiting a cohesion of just 1 kN/m^2 could stand with an exposed face to a vertical height of between 1 m and 2.4 m.) In this case, the plastic tank will not begin to carry significant load for a number of years. Creep in the plastic will be very low when there is high cohesion. Creep will increase as cohesion reduces and more load is taken by the tank. This will affect the overall time to failure and it is not just dependent on the creep strength of the plastic tank.

This is another reason why the actual earth pressure on a tank may be lower than predicted by simple analysis. This has not been taken account of in the reduction factor that has been developed in the following section.

Summary of the finite element analysis

A finite element model has been completed by G B Card and Partners to determine if the effects of arching can be allowed for in the design of geocellular tanks. The purpose was to reduce the design lateral pressure on tanks. The analysis has shown that at present, a conservative approach can be used to reduce the lateral pressure by 30% from the values predicted by Rankine earth pressure theory and those from the analysis of wheel loads following C737. The reduction can be applied when the following limiting conditions are met:

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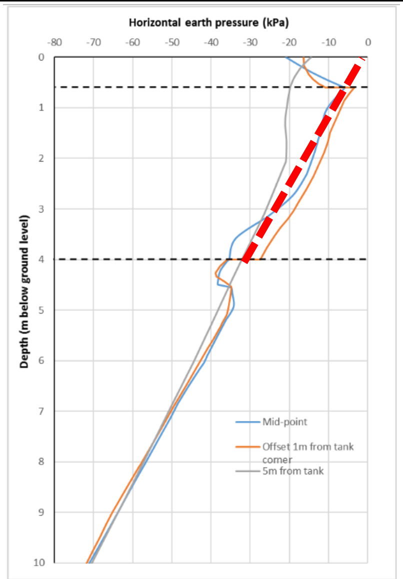
[8] H Perrier, Ultra Light Cellular Structures – French Approach. Geotextiles and Geomembranes 15, 1997, 59 - 76
[9] Technical Guidelines for Plastic Underground Storage and Infiltration Facilities (Draft). Association for Rainwater Storage and Infiltration Technology, 2008
[10] Le Nouveau N, Montaut M and Gomez A. Structures Alvéolaires Ultra-légères (SAUL) en Assainissement Pluvial: vers une Classification des Produits et Retours d'Expériences, Novatech, 2007
[11] BS 8002: 2015 Code of Practice for Earth Retaining Structures, BSI
[12] TRL Report 550, Analysis of the Stability of Masonry-faced Earth Retaining Walls, TRL, 2002

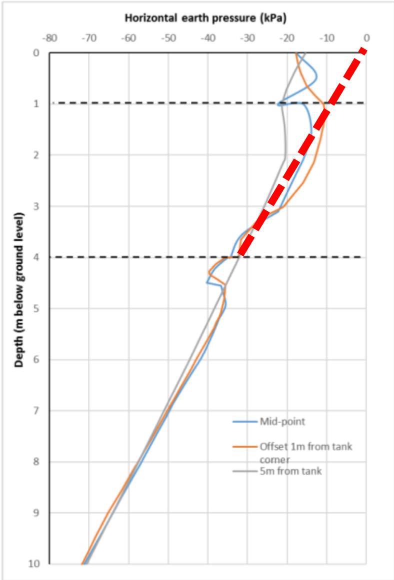
- The cover height to tank height ratio must be 0.48 or greater; and
- Soil to tank stiffness ratio must be 1.0 or greater.

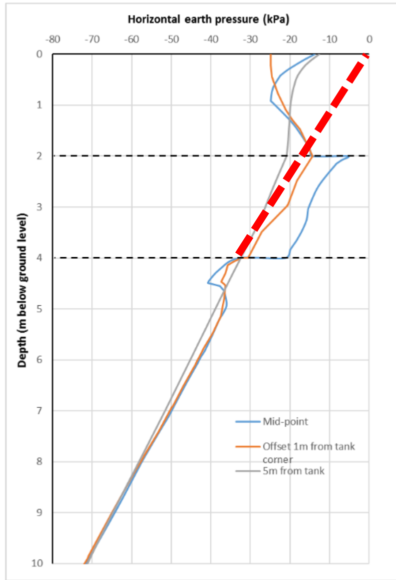
Further refinement and verification of the finite element model may allow much greater reductions to be applied in a wider range of conditions. Note that the reduction should only be applied to earth pressure and not groundwater pressure.

The results are summarised in the Tables below.

Table E.1 Soft clay with UDL



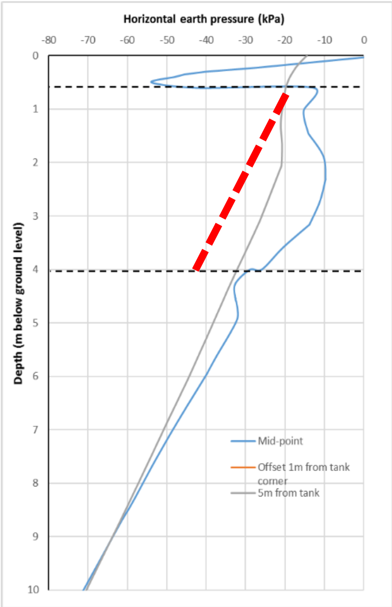


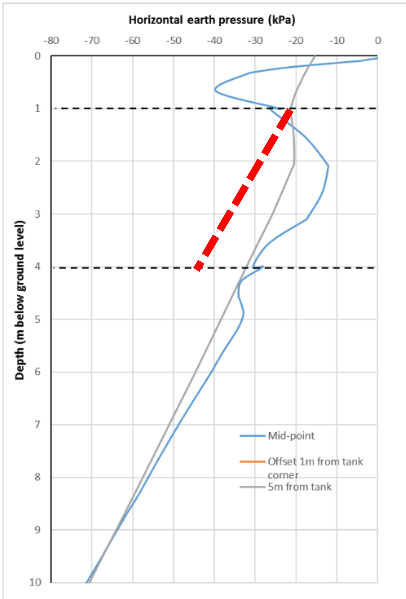


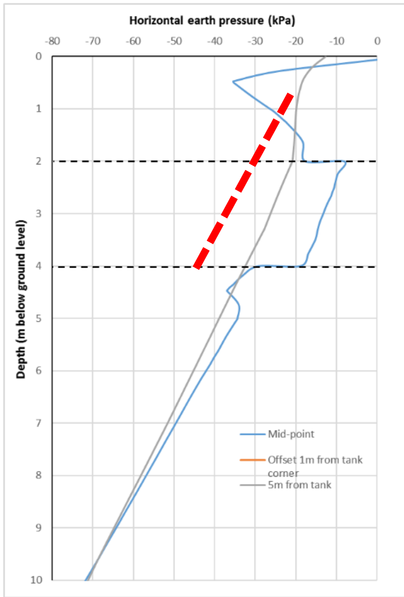
0.6 m cover depth	**1.0 m cover depth**	**2.0 m cover depth**
Soft clay, 10 kPa UDL, γ = 17 kN/m² ϕ = 24° K_a = TAN²(45 – ϕ/2) = 0.42 Rankine active earth pressure = 32.8 kPa (Red dotted line), (17 x 4 x 0.42) + (10 x 0.42)	Soft clay, 10 kPa UDL, γ = 17 kN/m² ϕ = 24° Rankine active earth pressure = 32.8 kPa (Red dotted line)	Soft clay, 10 kPa UDL, γ=17 kN/m² ϕ = 24° Rankine active earth pressure = 32.8 kPa (Red dotted line)

In summary, for a soft clay for cover depths of 0.6 m and 1 m, the earth pressure at the mid-point of the tank (horizontally) is effectively close to Rankine active pressure. When cover depth is 2 m, the pressure on the side of the tank at 4 m is 21 kPa which is a reduction of 11.8 kPa (36%).

Table E.2 Soft clay with wheel load



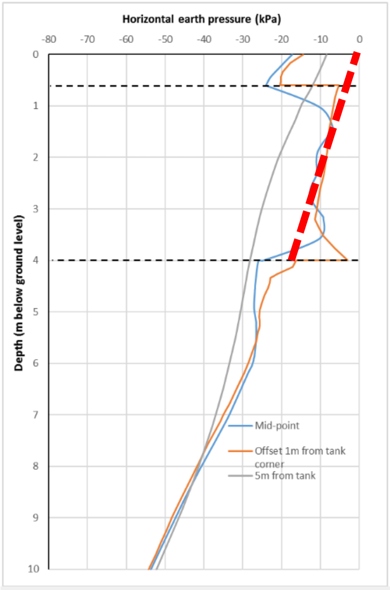
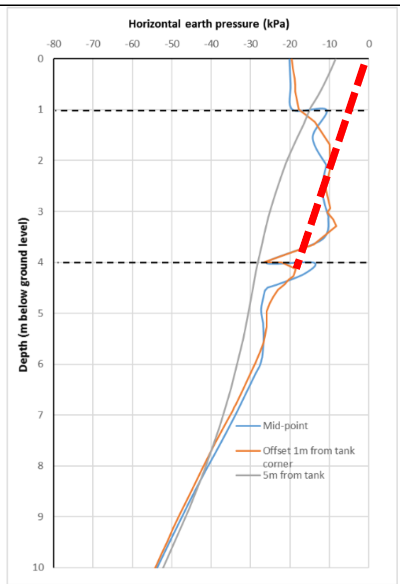
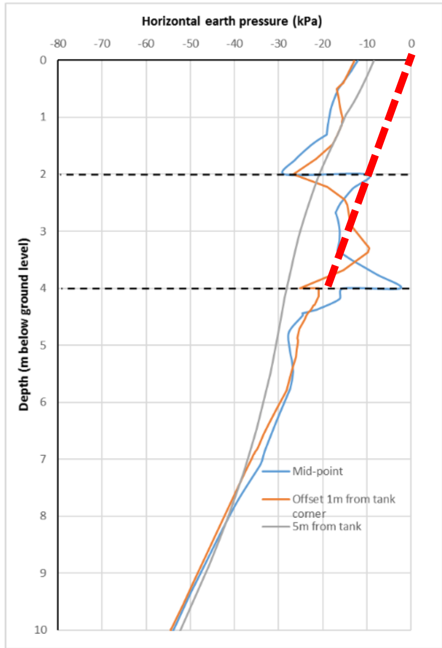




0.6 m cover depth	**1.0 m cover depth**	**2.0 m cover depth**
Soft clay Phi = 24˚ Earth pressure from wheel load C737 method = 14.78 kPa (Red dotted line) Earth pressure due to surcharge from wheel load C737 method = 14.78 kPa (Red dotted line shows total earth pressure)	Soft clay Phi = 24˚ Earth pressure due to surcharge from wheel load C737 method = 14.78 kPa (Red dotted line shows total earth pressure)	Soft clay Phi = 24˚ Earth pressure due to surcharge from wheel load C737 method = 14.78 kPa (Red dotted line shows total earth pressure)

In summary, the maximum lateral pressure from the wheel load occurs at shallow depth (approximately 0.5 m) which is consistent with analysis in the BPF Pipes Group guide to C737. The influence of the wheel load on lateral pressure below about 1.2 m cover depth is negligible. The total pressure on the tank at 4 m depth is 27 kPa for 0.6 m cover, 29 kPa for 1 m and 18 kPa for 2 m (38%, 33% and 58% reduction compared to values calculated using C737).

Table E.3 Dense sand and gravel with UDL

		
0.6 m cover depth	**1.0 m cover depth**	**2.0 m cover depth**
Dense sand and gravel, 10 kPa UDL, γ = 19 kN/m^2 ϕ = 40° K_a = TAN^2(45 – ϕ/2) = 0.22 Rankine active earth pressure = 18.9 kPa (Red dotted line), (19 x 4 x 0.22) + (10 x 0.22)	Dense sand and gravel, 10 kPa UDL, γ = 19 kN/m^2 ϕ = 40° K_a = TAN^2(45 – ϕ/2) = 0.22 Rankine active earth pressure = 18.9 kPa (Red dotted line), (19 x 4 x 0.22) + (10 x 0.22)	Dense sand and gravel, 10 kPa UDL, γ = 19 kN/m^2 ϕ = 40° K_a = TAN^2(45 – ϕ/2) = 0.22 Rankine active earth pressure = 18.9 kPa (Red dotted line), (19 x 4 x 0.22) + (10 x 0.22)

In summary, for cover depths of 0.6 m and 1 m, the earth pressure at the mid-point of the tank (horizontally) is effectively close to Rankine active pressure. When cover depth is 2 m, the pressure on the side of the tank at 4 m is 3 kPa which is a reduction of 15.9 kPa (84%).

Table E.4 Dense sand and gravel with wheel load

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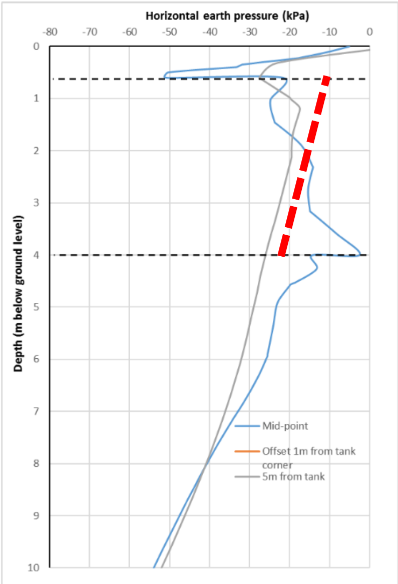
<td>
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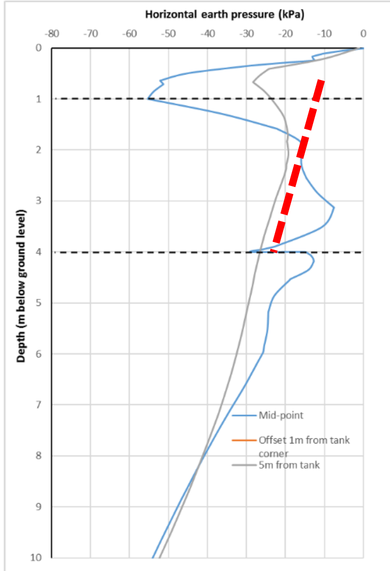
<td>
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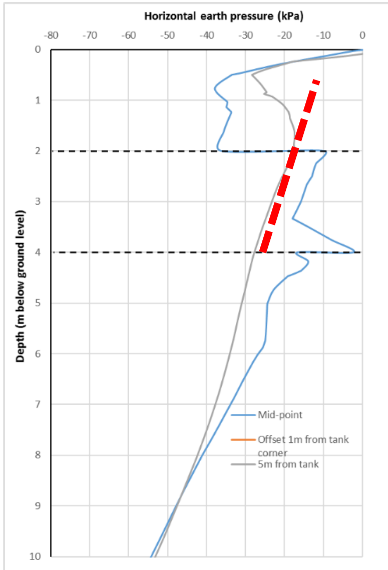
<td>
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<tr>
<td>0.6 m cover depth</td>
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<tr>
<td>Dense sand and gravel, wheel load
$\phi = 40^{\circ}$
Earth pressure due to surcharge from wheel load C737 method = 7.7 kPa (Red dotted line shows total earth pressure)</td>
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$\phi = 40^{\circ}$
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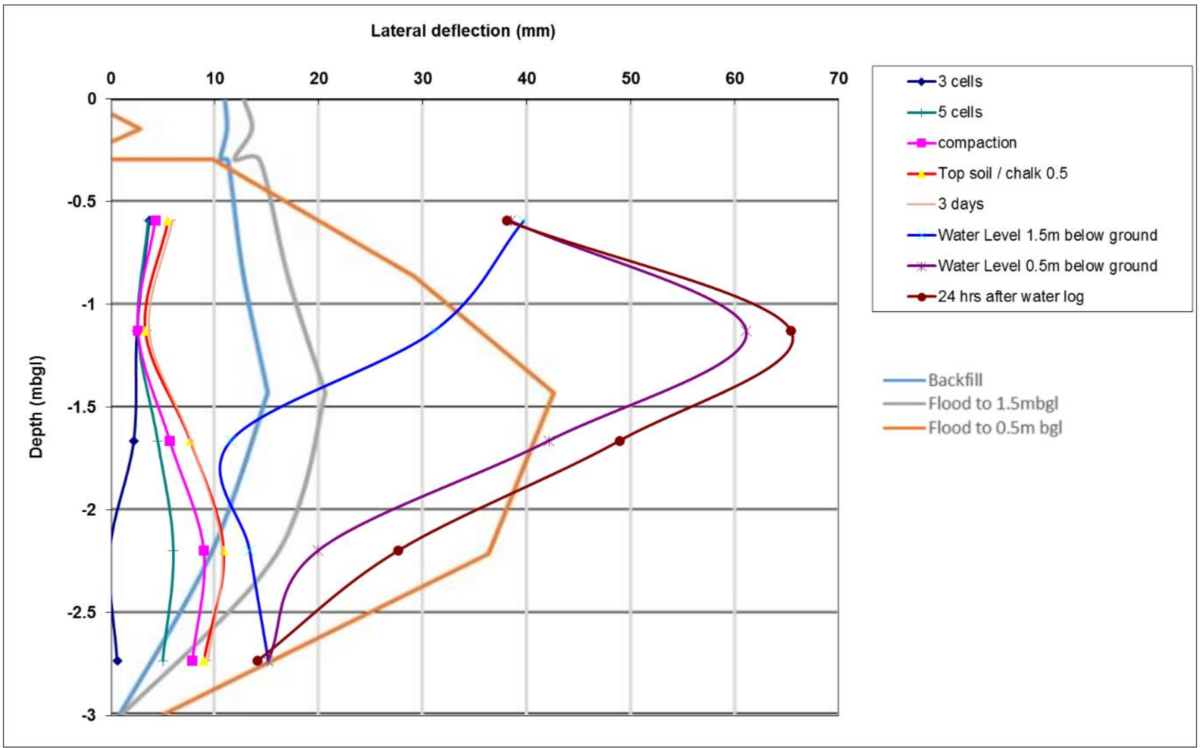
<td>Dense sand and gravel, wheel load
$\phi = 40^{\circ}$
Earth pressure due to surcharge from wheel load C737 method = 7.7 kPa (Red dotted line shows total earth pressure)</td>
</tr>
</table>

In summary, for dense sand gravel the maximum lateral pressure from the wheel load occurs at shallow depth (approximately 0.5 m) which is consistent with analysis in the BPF Pipes Group guide to C737. With 2 m cover, the peak load remains to 2 m depth and it is not clear why this is the case, although it makes no difference to the outcome of this assessment. The influence of the wheel load on lateral pressure below 2 m cover depth is negligible. The total pressure on the tank at 4 m depth is 3 kPa for 0.6 m cover and 2 m cover (88% reduction compared to values calculated using C737). The analysis suggests that there is a slight increase in lateral pressure for the 1m cover. This is not consistent with the other two analyses and is not considered representative in this assessment.

Ground truthing the model

The model has been compared to a trial installation that was completed at a site. The lateral movement of the tank was measured. The actual movement has been compared to that predicted by this finite element model.

Figure E.2 Comparison of predicted with actual movements



The measured deflections are lower than predicted with the soil backfill but are higher than predicted with the water pressure. These differences are likely due to the assumptions on soil properties made in the model. The variations in deflected profile are due to the simplifying assumptions on the lateral stiffness in the model. A uniform stiffness is assumed when in practice parts of the tank are stiffer than others. However, the results indicate a generally positive correlation between the model and the trial.

The earth pressure 5 m from the tank and below the base also fits well with the theoretical values.

Conclusions

For any tank/soil stiffness ratio, if the cover to tank height ratio is less than 0.5, there is no reduction in pressure for UDL. For cover to tank height ratio of 0.5 or greater, the lateral earth pressure with a UDL can be reduced by 36% from that predicted by the Rankine approach.

With a wheel load and dense sand and gravel, the maximum pressure on a tank with a wheel load can be reduced by 88%. For soft clay it can be reduced by 33%.

To derive reduction factors for use in routine design, a conservative approach has been adopted from the above analysis. For simplicity and to allow for some of the inconsistency in the results, assume a **30% reduction** from Rankine or C737 wheel pressure across all analyses where the ratio of cover depth to tank height exceeds 0.48 (see graphs below in Figures E.3 and E.4) and soil to tank stiffness ratio is greater than 1. This cover must be maintained below any services that cross the tank and measures should be put in place to prevent accidental excavation that would impair the arching effect.

Figure E.3 Earth pressure reduction against ratio of cover depth to tank height for surcharge

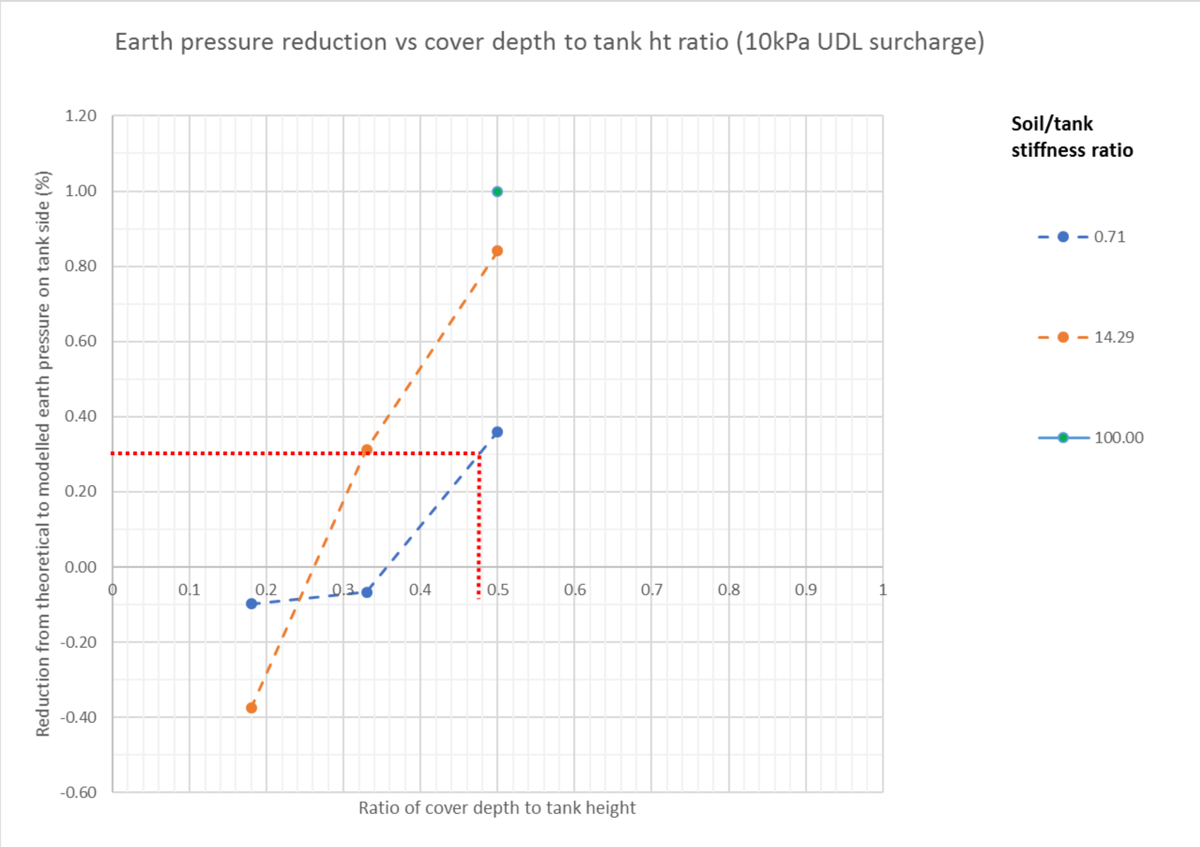
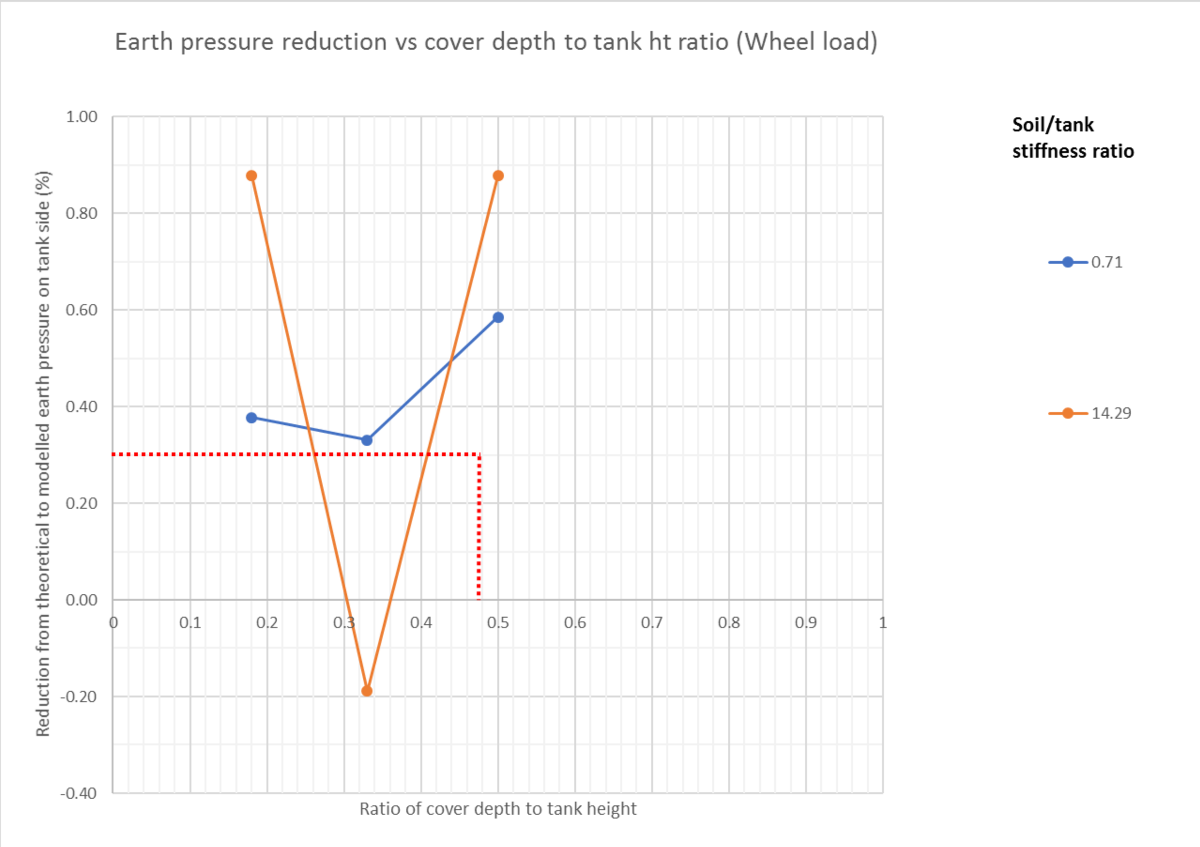


Figure E.4 Earth pressure reduction vs depth:tank height for wheel loads



Using a reduction factor to allow for arching may not be applicable where excavations for tanks are within the global critical shear surface for adjacent slopes or foundations. Arching effects apply only to soil and traffic loads and not to groundwater pressure.

With further modelling and verification of the model against field data, the graphs above could be refined.

The 30% reduction is consistent with the example in **Figure 2.39 of C737** (although the cover depth ratio for that example is not known). It is also consistent with the reduction in earth pressure on retaining walls using geofoam as reported by Koerner (2012)[7].

Appendix F: Overall design approach

The design approach is based on the standard concept that the sum of the factored load effects is less than or equal to the sum of the factored resistances.

$$\frac{\sum Load\ Effects}{Resistance} \leq 1.0$$

Equation (1)

As more than one type of resistance is involved (short-term and long-term) an interaction formula is used. A similar approach is taken in structural design if both bending and axial compression are being considered in a beam.

$$\left[\frac{\sum \gamma_i\ Q_i}{\emptyset R_n}\right]_{LS1} + \left[\frac{\sum \gamma_i\ Q_i}{\emptyset R_n}\right]_{LS2} + \cdots .. \leq 1.0$$

Equation(2)

where:

γ_i = load factors appropriate to the load considered;

Q_i = characteristic load for condition being assessed (long-term, short-term or hydrostatic);

R_n = characteristic resistance in the direction of loading appropriate to the condition being assessed;

ϕ = resistance factor for tank component appropriate to condition being assessed.

Equation (2) is used to complete the structural design in the vertical and lateral directions based on the design pressure and the structural resistance of the geocellular units. Equation (2) is specific to geocellular units and has replaced the terms for load, load factors, strength and strength factors used in Equation (1) with those used in C737.

$$\frac{Q_{dP}}{P_{dL}} + \frac{Q_{dT}}{P_{dS}} + \frac{Q_{dH}}{P_{dL}} \leq 1.0$$ Equation (3)

where:

Q_{dP} = design permanent load = Q_{ckP} x γ_{LFP} x γ_{sf}

Q_{dT} = design transient load = Q_{ckT} x γ_{LFT} x γ_{sf}

Q_{dH} = design hydrostatic pressure = Q_{ckH} x γ_{LFH} x γ_{sf}

γ_{LFP}, γ_{LFT}, γ_{LFH}, γ_{LFA}, γ_{sf} = Load factor (permanent), load factor (transient), Load Factor (hydrostatic), load factor (accidental) and site factor

Q_{ckP}, Q_{ckT}, Q_{ckH} = characteristic permanent load, characteristic transient load and characteristic hydrostatic pressure

Lateral loads have additional subscript of L, e.g., Q_{ckPL}

P_{dL} = Design characteristic long-term creep resistance from module tests = long-term creep resistance from module tests in the direction of loading appropriate to the design life, P_{ckL} x γ_{mL}

P_{dS} = Design short-term yield resistance from tests = characteristic short-term yield resistance from tests in the direction of loading, P_{ckS} x γ_{mS}

Lateral strengths have additional subscript of L, e.g., P_{ckLL}

γ_{mL}, γ_{mS} = resistance factor for tank component

Appendix G: Determining yield strength from short-term tests – the BBA approach

BBA have adopted the principle of determining the yield strength of units from short-term test data as follows:

1. Plot the load vs deflection on a graph (as shown in Figure G.1 below).
2. Ignore the seating part of the curve and any data beyond the peak failure load to determine the number of data points.
3. Locate the points on the load/deflection curve that correspond to 10% and 40% of the data points and draw a line between them – in the example below, that is the line with the equation y = 4.1636x + 5.2168. This is the trend in the elastic range.
4. Locate the points on the load/deflection curve that correspond to 90% and 100% of the data points and draw a line between them – in the example below, that is the line with the equation y = 0.3503x + 30.855. This is the trend in the plastic range.
5. The intersection of the two lines gives the yield strength – in the example below yield strength = 34kN.

Figure G.1 Example estimation of yield strength