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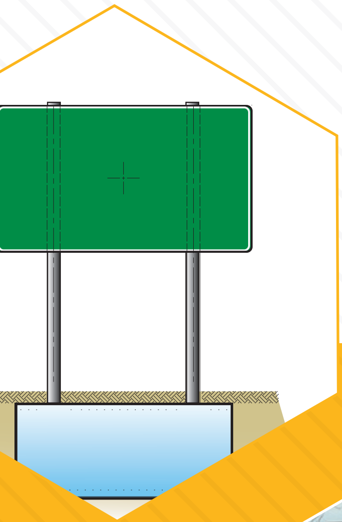
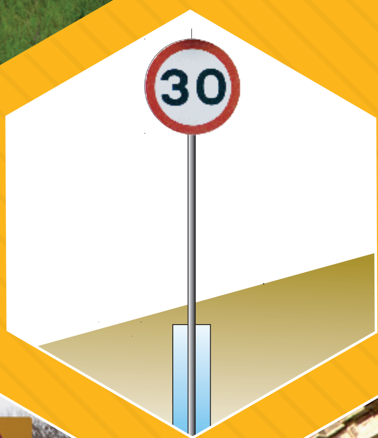
SIGN STRUCTURES GUIDE 2021

Support design for permanent UK traffic signs to BS EN 12899-1 and Eurocodes





January 2021

IHE SIGN STRUCTURES GUIDE: 2021 revision

In this edition foundation design has been extended to include guidance on the calculation and proving of the ground bearing pressures required in accordance with Eurocode 7. The document takes account of the 2019 update of the passive safety standard EN 12767. The implementation of trunk road standards CD 354 and CG 300 is covered for use where appropriate, with guidance given also on alternative methods that may be more economical in circumstances when these standards do not apply.

The Appendix C example calculations have been significantly extended. Those in example 1 now show planted foundation design to both the latest revision of CD 354 and to the lamp columns guidance document PD 6547, and the full working is given for a sign on a slope using both methods proposed in this Guide. Example 2 has been extended to include the design of both a reinforced concrete foundation and one using plain concrete. An example concrete specification is given.

In the **2010 edition**, passive safety was given greater coverage, foundation design became limit state in accordance with Eurocode 7, and guidance was added on wind funnelling and signs on slopes.

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Disclaimer

The Institute of Highway Engineers and the other contributors to this Guide have endeavoured to ensure the accuracy of its contents. However, the guidance and recommendations given should be reviewed in the in the light of the circumstances of each particular installation and specialist advice sought as necessary. This Guide is supplied on the condition that no liability for negligence or otherwise can be accepted by the Institute, its servants and agents, nor by other contributors.

For future editions, the IHE would appreciate being informed of any error identified or of any situation where following the guidance given herein has led to a problem in practice.

INSTITUTE OF HIGHWAY ENGINEERS

SIGN STRUCTURES GUIDE

Support design for permanent UK traffic signs to BS EN 12899-1:2007, Eurocodes and national standards

January 2021

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Editor	Simon Morgan, Buchanan Computing
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The example calculations and much of the commentary in the 2010 revision were prepared by Mott MacDonald for work commissioned by the Highways England. These have been updated in the 2021 revision by Highways England and other authors.

Arup Bridges and Geotechnics, Campus Midlands, have provided the majority of the 2021 updates to section 5 (Foundation Design) and the additional calculations in example 2, with further work by Highways England and Buchanan Computing.

About the authors

The authors below have been involved since the first edition of this guide in 2007, having worked also on the UK National Annex to BS EN 12899-1 and other UK and European standards and guidance.

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CONTENTS

1. INTRODUCTION

1.1 The design of supports for traffic signs has changed significantly as a result of the introduction of European standards. This booklet attempts to clarify the current situation and to provide a reference for all those involved with traffic signing. Much of Appendix C requires some understanding of structural engineering, but the bulk of this publication is intended for use by traffic and highway engineers: the people who most often need to specify sign structures. In conjunction with suitable computer software, it will cover most situations. It is nevertheless recommended that the advice of a structural engineer should be sought in cases of any doubt and always for larger signs and those mounted above the carriageway.

1.2 For many years sign structure design in the UK was standardised, a single wind pressure being used throughout the country. However, the relevant standard, BS 873, was withdrawn at the end of 2005, being replaced initially by BS EN 12899-1:2001 and now by BS EN 12899-1:2007. One of the changes from BS 873 is the need to specify what wind pressure each sign needs to withstand. There is thus an additional step in the design process that this Guide helps to explain.

1.3 The European standard offers a wide range of classes for each aspect of a sign's performance, in order to address the requirements of all participating countries. To assist designers of traffic signs in the UK and to encourage consistency where it is appropriate, the UK version of the standard, BS EN 12899-1:2007, has a National Annex appended. This contains recommended values and classes for the options, including for wind loading, as is therefore an invaluable reference.

1.4 The determination of the wind load and the design of the structure are two essentially separate tasks that do not necessarily need to be undertaken by the same person or organisation. It is recommended that when a wind load (or *wind action* to use the terminology of the Eurocodes) is recorded or communicated between organisations as part of a sign specification, this should be the basic wind pressure, w_b. This is the load prior to the application of any safety factor or force coefficient, and is equivalent to and comparable with the values in Table 8 of BS EN 12899-1:2007 and table NA.2 of its National Annex. As the nine WL classes in BS EN 12899-1:2007 differ from those in the previous edition, the use of the basic wind pressure is recommended in preference to stating a WL class to avoid any confusion as to which version of the standard is intended, and to permit the range to be extended and intermediate values specified.

1.5 The UK National Annex to BS EN 12899-1:2007 includes a table of suitable wind pressures, broken down by country or region and overall sign height. It does not address signs over 7 m total height, nor those at an altitude of more than 250 m above sea level. This advice is the result of work commissioned by the Highways Agency (now Highways England) and significant discussion amongst industry experts. The intention was to produce guidance that was (a) reasonably simple to use, (b) complied with current standards on wind actions and (c) resulted in support sections that were comparable with those typically specified under previous standards. Inevitably this requires a much more detailed appraisal of the sign and its location than using a blanket wind load value. This booklet explains how to use this new method.

1.6 Those involved in designing the faces of traffic signs, particularly directional signs, should be aware that they frequently have a choice of layout and that the size and cost of the supporting structure will depend upon how they use this freedom. For example, a tall narrow sign requires stronger posts and foundations than a 'landscape format' sign of the same area. Even where the available width is constrained, preventing any major rearrangement of the layout, it is often possible to reduce the sign face area by altering the positions of text and symbols to eliminate some of the blank space. In certain situations, splitting directional information across two separate signs in advance of a junction is preferable to using a single large sign, both for driver perception and structural reasons. However, when attempting to reduce the size of a sign, on no account should the size of lettering or the layout guidance in the Traffic Signs Manual and DfT working drawings be compromised, as this would degrade the readability of the sign and make it less capable of fulfilling its purpose of helping the road user.

Sign structure design

1.7 Sizing the supports for a sign is a matter of balancing risk and economy. The most obvious risk is that a traffic sign structure might fail in strong winds, leading to the economic cost of replacing it and the safety and other disbenefits of the sign being absent in the meantime. There is also the danger that a failed or failing sign structure might cause damage or injury as it falls or as a result of it obstructing the highway.

1.8 However, the risk associated with the failure of a properly designed sign structure is negligible compared to the risk that a vehicle might leave the carriageway and collide with it. For this reason, passively safe (or crash friendly) supports are increasingly specified for use on classified or well-used roads. A now superseded Design Manual for Roads and Bridges (DMRB) standard required safety fence protection or passively-safe supports on roads with a 50 mph or higher speed limit, but the National Annex to BS EN 12767:2019 recommends that passive safety should be a consideration on all roads. Passively safe signs must still meet the structural requirements of BS EN 12899-1, although a greater deflection is permitted. Passive safety is considered in more detail in Section 2. There is always a safety benefit as well as an economic one in using the most slender support that is structurally adequate.

1.9 Over-sized supports cost more, add to visual intrusion and are more likely to lead to death or injury than correctly designed ones. Over-design may also result in unnecessarily large foundations, leading to a longer construction period and thus greater disruption, coupled with a greater difficulty of locating such bases in urban roads containing extensive services. It is therefore well worth the trouble of designing sign structures using the method described in this publication, which aims to minimise support sizes whilst adhering to the relevant standards and sound engineering practice.

Wind actions

1.10 The most significant force on a sign is that exerted by the wind. Determining wind actions involves an element of probability and some knowledge of the physical characteristics and topography of the location concerned. EN 12899 uses the term *wind load*, which is synonymous with *wind action*.

1.11 The relevant standard for wind actions is BS EN 1991-1-4:2005+A1:2010, which is referenced by BS EN 12899-1. The UK National Annex to BS EN 1991-1-4 includes a map of the UK showing the base reference wind speed for any location. To this value a number of corrections need to be applied to allow, for example, for the altitude, the height and ground clearance of the structure, its proximity to the coast or large inland lake and whether it is within a town or rural situation. Consideration also needs to be given to any local funnelling effect that might occur between tall buildings in a town environment, in a valley with a significant narrowing, or whether there is a sharp rise in ground level close to the relevant location. It is thus time consuming and generally uneconomic to derive an accurate wind load assessment individually for each sign location.

1.12 The National Annex to BS EN 12899-1:2007 recommends suitable wind loads for the majority of signs in the UK. Whilst is it much more conservative than performing a full analysis, it is simpler and quicker. The method is not suitable for all situations: it cannot be used at an altitude of greater than 250 m, or where there is significant wind funnelling or topographical features such as cliffs or escarpments that affect the wind flow. Locations with these effects are referred to in the Eurocode as having significant *orography*, and the recommended action is to revert to BS EN 1991-1-4. This Guide consequently gives examples of both methods of calculation. A detailed wind load determination using BS EN 1991-1-4 is recommended whenever possible, as it can often result in a load as little as half that indicated by BS EN 12899-1.

1.13 At present there is no published guidance available for design in areas of wind funnelling. Section 6 contains interim guidance on this subject.

1.14 The 12899 National Annex suggests that authorities responsible for signs within a defined area may wish to derive a wind load that can be applied to all their signs, or to sub-divide their area into regions with similar wind characteristics. This will result in significant economy for areas located towards the south or east of their country or region or where the maximum altitude of any public highway is well below the 250 m assumed in the National Annex. It will also make subsequent design work easier for authorities that have areas for which the simplified method cannot be used. The work involved in applying the methodology of BS EN 1991-1-4 a single time will be amply repaid for every sign that is subsequently designed to the wind load thus obtained.

1.15 In practice, most designers of sign structures will use appropriate computer software. They nevertheless need an overview of the methodology involved to ensure that they are using the software correctly, with suitable options and parameters selected, and to check that the results obtained are reasonable. A reference to software available free of charge is given in Appendix D.

Other actions on signs

1.16 Other forces that may need to be taken into account when designing a sign structure are point loads and dynamic snow load. The UK National Annex recommends that signs should be able to withstand a force of 500 N applied at any point. This represents the load that might be exerted by, for example, a glancing blow from a vehicle mirror, a falling branch or malicious interference with the sign. This point load is the critical factor only for very small signs, but for signs mounted on a single support it causes torsional forces that need to be considered.

1.17 Dynamic snow load is relevant only to low-mounted signs in areas where snow ploughs or snow blowers are regularly used if there is a significant problem of sign damage caused by these operations. The BS EN 12899-1 National Annex recommends that class DSL1 (1500 N/m^2) is relevant to snow blowing and class DSL2 (2500 N/m^2) to snow ploughing at up to 40 mph. As snow clearance is likely to be regular only in areas of high wind, and the dynamic snow load is not applied to any portion of the sign higher than 2.5 m from the ground, it will be unusual for the snow load to be the critical factor in sign structure design. It is therefore rare for snow loading calculations to be required.

Comparison with previous methodologies

1.18 Prior to the introduction of BS EN 12899, the structures for most small and medium-sized traffic signs were designed using the DfT nomograms or a computer simulation of them. These charts with the reference WBM140 were first produced for the 1967 edition of Chapter 13 of the original Traffic Signs Manual and were last updated in 1983. They continued to be used, despite being based upon superseded British Standards, as there was no better guidance readily available.

1.19 The nomograms provided a choice of wind pressures and Chapter 13 recommended 1530 N/m^2 for signs in exposed positions and 960 N/m^2 for signs in urban areas and sheltered places. This pressure was rounded to 1500 N/m^2 in later revisions of the former traffic signs standard, BS 873. A comparison between the methodology of BS 873 and the BS EN 12899-1 National Annex shows that broadly similar sections are achieved in England, but that in Scotland a larger section than would have been used previously is generally indicated. The difference for signs on a single post is particularly marked because the nomograms used a more onerous deflection criterion for signs on more than one support.

1.20 BS EN 12899-1 and BS EN 1991-1-4 require the use of more conservative force coefficients and partial action (safety) factors than BS 873 or the nomograms. Hence, for a typical small sign, the BS 873 standard wind load of 1500 N/m^2 results in a broadly similar size of support section to a load of 1100 N/m^2 using the current method of calculation.

2. PASSIVE SAFETY

Introduction

2.1 Traffic signs, lighting columns, and other highway structures provide a useful function in making the highway safe and usable and providing driver information. However, designers should be aware that objects installed by the roadside pose a risk of injury to the occupants of any vehicle that leaves the carriageway. Passively safe structures are designed to provide less resistance during impact reducing the severity of that impact for the occupants. The performance levels are confirmed by testing.

2.2 All structures need to comply with the structural requirements, which for traffic signs is BS EN 12899-1. Passive safety is an additional characteristic – it does not override the need to ensure that the structure can support the required loading. The only relaxation is in the deflection requirements detailed in BS EN 12899-1 NA.2 (extract included in Appendix A).

Why use passively safe posts?

2.3 The photographs below indicate the different levels of damage incurred in similar impacts. The traditional post would have resulted in probable death for vehicle occupants whereas it is unlikely that any injuries would have been incurred hitting the passively-safe post.

***Traditional post** (114mm wide base)*
vehicle badly damaged, rapid deceleration

***Passively safe post** (140mm dia)*
minor damage to bonnet/bumper
vehicle still driveable, low deceleration.

Costs

2.4 Passively safe posts are generally more expensive than the equivalent steel post. This may influence the decision to specify them, but this must be balanced against the significant reduction in risk. Other factors may have to be considered. The omission of vehicle restraint systems can reduce scheme costs. Electrical disconnection systems (where required) can add costs but they allow rapid replacement of the post if struck, reducing the disruption and costs. This may be an issue at certain sites. As with every scheme, this is a balance between available budget and the benefits of the various risk reduction measures.

Where to use Passively Safe Structures

2.5 Passively safe structures have been used predominantly on high-speed roads where the benefits are significant. In urban situations where speeds are lower they still have benefits, but the presence of obstructions such as buildings and parked cars, and anything that restricts vehicle speeds, such as traffic calming and road geometry, will reduce the benefits. In these situations a collision with the structure in question at a speed sufficient to cause it to behave passively becomes less likely, and there may be insufficient room for the structure to deflect if adjacent to a wall or building.

2.6 Passively safe products are part of the wide armoury of tools available to improve road safety. Designers need to be aware that they will not be the best solution in all circumstances. Limitations on budgets restrict possible solutions, so designers will need to assess the optimum use of available funding. Simple measures like renewing white lines or using anti-skid treatment may be preferable where larger areas can be treated for the same budget. But when used effectively to reduce the risks at specific locations, identified by accident records or risk assessment, passively safe structures allow the benefits of having the sign in the desired location whilst reducing the risk associated with a traditional post.

Risk Assessment – Individual Structure

2.7 When assessing requirements for any site it is recommended that designers compile a number of queries that need to be addressed. Some typical questions/procedural steps are listed below:

1. Is the structure really needed? Is it required for safety reasons or to comply with legislation or best practice, or has the safety audit identified this as necessary?
2. Does it have to be that size and weight? Directional signs can often be made smaller (without reducing the size of lettering) with careful layout changes.
3. If it is essential, can it be relocated to a safer position?
4. If not, consider a passively safe structure.
5. If there are other hazards, like bridge piers, that need protecting with a Vehicle Restraint System (VRS) use traditional sign supports.
6. Where it is proposed to install a sign behind a VRS protecting an existing structure and it is likely that an existing structure will be removed in the near future, negating the need for VRS. Then consideration should be given to installing a passively safe structure, as this will allow the VRS to be removed without any additional need for protection of new structures.

2.8 Risk assessment is the principle that must underpin all projects. Designers are required to demonstrate that they have a robust assessment method for determining risks associated with each scheme, which incorporates a methodology for evaluating improvements to existing hazards against associated costs of incorporating this within the scheme. Designers should commence with the objective of minimising all street furniture. If there are no obstructions on the verge, then the risks to the driver of an errant vehicle are significantly reduced. The use of passively safe structures normally allows installation without the need for VRS, which is of course itself an obstruction. So, along a typical length of carriageway there will be fewer objects to hit and the probability of a collision will be reduced. Individually passively safe structures are more expensive than traditional steel posts however the omission of VRS can realise significant cost savings on most schemes in addition to reducing risks.

Primary Risk

2.9 This is the first of two main considerations. When a vehicle strikes a structure, such as a lighting column, at moderate to high speed, it is probable that the car will suffer serious structural damage. During a high-speed impact, even if the car remains relatively undamaged, the forces on the internal organs of vehicle occupants can cause fatal or serious injuries, despite the use of seat belts and air bags. By altering the properties of the column, the consequence of a vehicle strike can be greatly reduced. This is the principle behind passive safety and, correctly applied, does significantly reduce the risks to the vehicle occupants.

Secondary accidents

2.10 The second consideration is the possibility that, after the initial impact, the vehicle will continue unrestrained leading to a further accident, or that the debris will cause injury to other road users.

2.11 These two risks cannot be considered equal. A study of the behaviour of structures during crash tests indicates that the 'debris' will generally fall back over the vehicle at high speed and forward at low speed, and in either case be deposited close to its original position. Therefore, where a structure is situated on the verge the probability is that debris will remain on that verge. For a secondary accident to occur the post has to be struck and fall, or be dragged onto the carriageway, and then be of sufficient size to cause damage or an accident. This also assumes that the oncoming vehicle cannot stop or avoid the debris. Risk assessments should recognise that primary and secondary accidents are a different order of probability: secondary accidents are very rare occurrences and therefore a very low risk.

Risk Assessment – Global environment

2.12 It is important that designers consider the whole roadside, not just an individual structure. This is particularly important in small projects where only one or two structures will be replaced, but other existing structures will remain. The full benefits of expenditure on passively safe structures will only be realised if all the risks associated with the location are considered and mitigated. Drivers make mistakes and the ultimate aim for all designers is to provide a roadside that is more tolerant and forgiving.

2.13 All structures pose a risk to drivers during impact. Traditional structures present a much greater risk than passively safe structures. If designers install passively safe structures on a roadside without considering the adjacent structures they should be aware that their proposals could increase the risks associated with that stretch of road if they have simply added additional potential hazards without addressing the underlying problem. This may be difficult to justify in a road safety audit.

2.14 Installing a new passively safe sign close to, say, a traditional unprotected lighting column undermines the principles of EN 12767. Every attempt should be made to address existing hazards within any new scheme. A major cost on most schemes is traffic management, for a minimal additional cost some hazards could be removed. Designers should seriously consider the risks associated with leaving existing hazards untouched. It is the designers' responsibility to assess the risks associated with each proposed scheme and devise a solution that best addresses those risks. It is recommended that all decisions are recorded especially where existing hazards are not being addressed.

Performance Classes

2.15 UK engineers are increasingly specifying passively safe products. However, it is not sufficient to specify any product without having an appreciation of the differences between product classifications, and the impact speed for which they have been designed and tested. For structures to be declared passively safe they must be crash tested in accordance with EN 12767. The results of these tests will determine the classification of the post. There are three main categories of passively safe structure:

- high energy absorbing (HE);
- low energy absorbing (LE);
- non-energy absorbing (NE).

The National annex to EN 12767 describes the distinction between the energy absorbing and NE classes thus:

> *Energy absorbing support structures slow the vehicle considerably and thus the risk of secondary accidents with structures, trees, pedestrians and other road users can be reduced.*
>
> *Non-energy absorbing support structures permit the vehicle to continue after the impact with a limited reduction in speed. Non-energy absorbing support structures may provide a lower primary injury risk than energy absorbing support structures.*

Posts that are tested and do not comply with the requirements or are not tested can be classified as *Class 0 to EN12767*, but Class 0 posts are not passively safe.

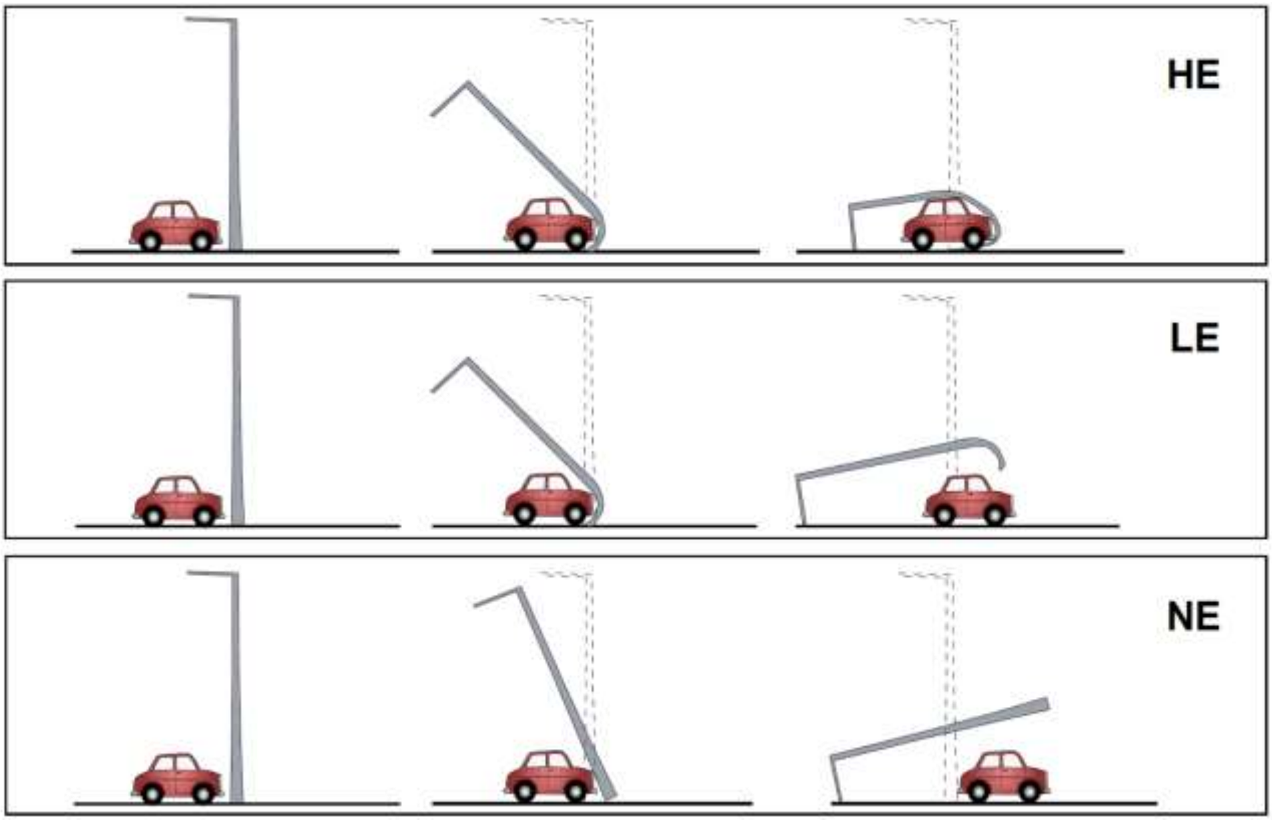
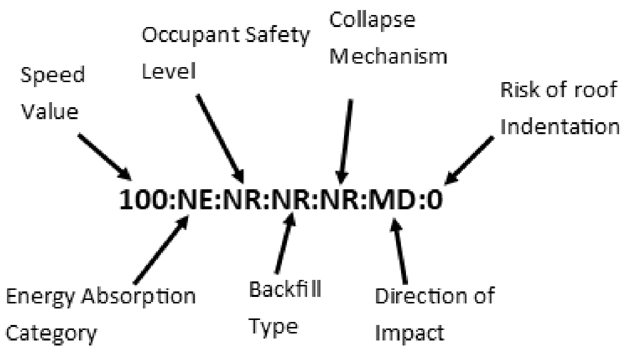


Figure 2.1 Typical failure modes to be expected for different energy absorption classes

2.16 Non-Energy absorbing structures (NE) provide the lowest risk to the vehicle occupants, but the vehicle will travel further and may hit another object. With High Energy absorbing posts (HE) the vehicle will be decelerated to under half its initial speed, but the risk to its occupants increases. There is no one solution suitable for every scheme, but passively safe supports for traffic signs are overwhelmingly of the NE class.

2.17 The speed value (100, 70 or 50), placed before the energy absorption class, indicates the maximum speed (km/h) at which the product has been tested, and should accord with or exceed the known or measured 85th percentile speed of traffic on the road (not necessarily the speed limit). All products have also been tested at 35km/h. The performance class of each structure is expressed as an alpha-numeric string that sets out the individual performance for specific aspects, where the text is shown as NR this means that there is no requirement for this class.

2.18 The most common specification of EN 12767 classes for a traffic sign support in UK is:



Which means a support tested at 100 km/h that is non-energy absorbing can have any occupant safety level, any backfill type, any collapse mechanism, can perform in any direction and has a roof indentation of less than 102mm. The options for each class are shown in table 2.1.

	Speed class	Energy absorption category	Occupant safety class	Backfill type	Collapse mode	Direction class	Risk of roof indentation
Alternatives	50, 70, 100	HE, LE, NE	A, B, C, D, E	S, X, R	SE, NS	SD, BD, MD	0, 1

Table 2.1: All possible alternatives for each field in the passive safety performance class

Practical use of passively safe products

2.19 An understanding of the performance characteristics of different passively safe products can assist designers, but some caution is needed to ensure that an appropriate solution is proposed.

2.20 At locations with a high density of non-motorised users (NMUs) some designers have proposed High Energy posts in the belief that these would slow the vehicle and prevent secondary accidents. What must be remembered is that it is not a normal function of a traffic sign to restrain errant vehicles. Were a sign no longer to be needed, its supports would not be retained just for their restraining effect. A sign is unlikely to be in the right location to protect a NMU. Therefore, alternative protection should be considered if there is a significant risk to NMUs. The test procedure for EN 12767 involves recording the exit speed at a distance beyond the structure: for a 12 m lighting column this would be 12 m in the direction of travel. Even at that position, a 100:HE post would have an exit speed of up to 50 km/h, which might still present a significant risk to NMUs. The designer therefore needs to balance the risk to vehicle occupants in achieving the greatest reduction in speed against those to NMUs in not sufficiently restraining errant vehicles.

2.21 Urban roads with a high NMU density generally have speed restrictions, so the risks are reduced. Other considerations are roads where the density of daytime traffic limits maximum speed, and conversely that many single vehicle collisions with roadside objects occur at night when fewer NMUs are about.

2.22 Locations such as nosings, slip roads, roundabouts and central reserves need careful consideration. Safety barriers may not be suitable where, for example, the impact angle would be too steep, or the barrier could block visibility. A passively safe support without further protection is often a satisfactory design solution. The slight increase in the already small risk of secondary accidents is outweighed by the significant benefit of being able to install a sign or signal at the optimum location. Risk assessments for traffic signal installations justify the use of passively safe posts for most locations on this basis.

2.23 For high-speed dual carriageways and motorways it is likely that an existing vehicle restraint system will already be in place, and it is important to take into account that they deflect when struck. The working width is the zone from the traffic face of the barrier, before impact, to the extreme position of any part of that barrier on impact. This varies depending on the specific VRS (see table 4 of EN1317-2). If a structure were installed within this working width it would restrict the deflection of the barrier during impact, significantly increasing the risk to the driver. Also, there would be a risk of more damage to the vehicle and of a secondary accident, as the vehicle might not be deflected back onto the carriageway but put into an uncontrolled spin.

2.24 Serious reflection should be given to any suggestion that proposes installing a structure close to the barrier. Designers must justify the need for that structure at that particular location. In exceptional circumstances consideration may be given to small structures that would provide minimal resistance during impact. Some guidance is given in CD 377: Road Restraint Systems (DMRB Volume 2, Section 2, Part 8).

Certification

2.25 Designers must satisfy themselves that the product selected meets the requirements of their design. This may be achieved by certification; third party checks or specific checks to the client requirements.

2.26 Where products carry a CE or UKCA mark, the properties of the product will have been verified by a Notified Body and can be accepted without further checks, provided the classes to which the product conforms accord with those specified.

2.27 Without CE or UKCA marking, designers have to examine the test reports to compare the results with the standard and determine acceptability for every proposed product. To ensure compliance with EN12767, a designer requires an understanding of the test procedures and the way the results are reported. Many test reports do not provide a summary and the designer needs to be able to interpret the results to determine the suitability of any particular product.

2.28 In order to avoid this necessity, a system of 'third party checks' has been used by many manufacturers and accepted by client organisations. To achieve this, an independent body or person, such as a consulting engineer, examines the documentation and verifies compliance.

Steel posts

2.29 The Highways Agency (now Highways England) carried out testing to demonstrate that small steel posts are passively safe. This work has been incorporated into BS EN 12767. These are an exception to the requirement that individual products should be tested, as generic approval (deemed to comply) has been given irrespective of manufacturer. Clause NA 4.1 and Annexes A and K record that posts of 89 mm diameter and 3.2 mm wall thickness or smaller (of steel grade S355J2H or lower) are passively safe, and 76 mm diameter posts can be used at 300 mm centres (or 750 mm centres if three or more supports are needed). Annex K states that the results are also valid for and steel or aluminium sections of 89 mm diameter or smaller with a lower bending and shear capacity than the support tested. Many standard steel and aluminium supports are therefore considered passively safe, in addition to proprietary products. Whilst the testing of these sections used a mounting height of 2.1 m, signs of a size typically installed on these smaller supports are unlikely to cause windscreen penetration in a collision, so lower mounting heights are permissible. In any case, the failure mode observed in testing was for the post to fold over at the base on impact, avoiding any possibility of windscreen intrusion.

2.30 Standard sections can be a cost-effective alternative to products specifically marketed as passively safe. They are generally only suitable for smaller signs, but in an urban location this would cover many of those used. Sufficient capacity for their use can frequently be obtained by using the more detailed and site-specific wind loading calculation method of BS EN 1991-1-4, described in this Guide

Interpreting the standard

2.31 EN 12767 is specific in the requirement that a product is tested for the intended use. A post tested as a lighting column cannot be assumed to be passively safe if it supports a speed camera. If a product was tested with a thin aluminium sign plate it must not be assumed to be acceptable for supporting a variable message sign (VMS). The structural capacity of the post may be adequate for other applications, but without the test data it cannot be marketed as passively safe.

2.32 However, risk assessment may be utilised to consider using products in demonstrably similar or less onerous circumstances not fully covered by available test data. For example, a post tested supporting a very large sign could be considered suitable to support a small CCTV camera. There are no guidelines that can be given as an acceptance criterion, so the designer has to justify these by demonstrating that the benefits are considerably greater than the risks.

2.33 For sign supports it is normal to test the structure with a sign mounting height of 2m, although a number of proprietary supports have now been tested satisfactorily at 1.5m. This height of 1.5m is common in UK, as it accords with Department for Transport (DfT) recommendations for rural areas, and a higher mounting height would require larger supports increasing the risks. On most cars the windshield is generally between 1.5m and 2m. The lower the proposed mounting height, the greater the risk to the driver that the sign or its supports will puncture the windshield. Designers have to consider the risks associated with specifying a lower mounting height than the proposed product has been tested to. For example, a VMS or other heavy sign at a low mounting height carries a considerable risk of puncturing a windshield on impact, whereas a light gauge aluminium plate or composite material with small channels has an acceptable risk.

2.34 The spacing of two or more posts must usually provide a clear opening of 1.5m at the impact angle of 20º unless the supports have been tested specifically for impact on more than one post, or standard steel sections are being used. Enhanced aluminium channel sections are available to stiffen the sign face allowing a wider than usual span between supports or a wider overhang either side of a central post. However, confirmation should be sought from the supplier that this will not affect the performance during impact. Use of such sign plates helps to achieve the necessary spacing, but positioning passively safe posts remains a challenge, particularly where there is a footway or cycleway than cannot be obstructed.

2.35 Where a sign in a cutting has multiple posts, designers have queried whether the measurement of mounting height should be taken at the nearside post or the one furthest from the carriageway. The back post is further from the carriageway, so the probability of impact is less. Many designers have rationalised that it may be acceptable to consider a slight increase in the risk of a lower mounting height at the back post, as this can be balanced against the reduced chance of an impact.

2.36 Making the road environment more forgiving is an aspiration for all designers and will have significant safety benefits. The use of passively safe structures can be an effective solution to reducing the risks associated with vehicle impact. These structures have different requirements to traditional steel posts, which designers will need to understand and take account of. This may require appreciation of the general principles, specific knowledge of the product as well as a global perspective on all the available options.

2.37 Road safety is an important issue with a high public profile. Government targets are challenging. The available budgets are being stretched so designers have to constantly review and adapt possible solutions to maximise benefits. Passively safe structures can provide a cost-effective solution whilst minimising risks. It is recommended that designers familiarise themselves with the basic principles to allow more effective consideration of improvement options.

Further information

2.38 The publication *Passive Safety UK Guidelines for Specification and Use of Passively Safe Street Furniture on the UK Road Network* provides guidance for designers. The Passive Safety UK website (see Appendix D) has this available for download, together with background information and lists of available products.

3. BACKGROUND INFORMATION ON WIND ACTION

General

3.1 The National Annex to BS EN 12899-1:2007 provides wind load values for the UK as an alternative to calculating actions using BS EN 1991. The wind pressure obtained using the table NA.2 (Appendix A) is conservative so, for large signs or where a number of signs are to be placed in a similar location, it is recommended that detailed calculations to BS EN 1991-1-4 be carried out.

3.2 The simplified method using the National Annex wind action values will take between ½ to ¾ hour depending on the complexity of the sign and location. This compares favourably with the alternative using BS EN 1991-1-4 to calculate the wind action, which can take several hours. The EN 1991-1-4 method is more complex and requires more detailed information about the location of the sign, which may not be readily available to the designer. It is possible to put much of the work into a spreadsheet to significantly reduce the design time, or to use suitable software that is available. It is also possible to calculate a wind load that can be safely applied to a whole scheme, authority or maintenance area to avoid repeating this work for every sign.

3.3 In the examples the partial action factor recommended in the 12899 National Annex (class PAF1) is applied and an additional factor γ_{f3} of 1.0 included. γ_{f3} is traditionally used to allow for the possibility of inaccurate assessment of the effects of actions and unforeseen stress distribution in the structure. For a simple cantilever sign structure, the possibility of inaccurate assessment of the effects of actions is limited, and, where the posts have been verified by static load testing, unforeseen stress distribution in the structure and variations in dimensional accuracy achieved in construction are also unlikely. Similarly, for steel sections the reliability of the product is considered equivalent to static load tests so a factor of 1.0 is proposed. A designer may use this factor to allow for the possibility of future modifications to the sign or the addition of small signs to the post.

3.4 Designers will be able to achieve more efficient designs using the more rigorous method, but the proposed National Annex simplified method will minimise the design effort for all but the more complex situations.

3.5 The example calculations are set out to match the format of limit state design codes. Load action $\times$ load factors $\leq$ element capacity/material factor.

3.6 The BS EN 1991-1-4 National Annex methods should be used for calculating the exposure factor, $c_e(z)$, using a look-up table based on the distance from the shoreline. The wind action also depends on the height of the centroid of the sign from ground level. The look up wind loads presented in 12899-1 National Annex Table NA2 are based on this method, with the assumption of terrain category II. This terrain category is valid for all but the most exposed situations, but will be conservative for sheltered town sites.

Aerodynamic Force Coefficient, c_f

3.7 The National Annex to BS EN 12899-1 Table NA.2 (Appendix A) gives the force coefficient (also known as the shape factor) for elements with different aspect ratios. This is based on a review of the available guidance and how it should be applied to signs. EN 12899-1:2007 specifies the use of c_f = 1.20 (clause 5.3.1.1). This conflicts with advice in BS EN 1991-1-4 clause 7.4.3 which proposes a value of c_f = 1.8 for 'signboards'. The value of 1.8 is very conservative for signs, and it is valid to use an alternative method that considers them as structural elements with sharp edges.

3.8 Two different methods were reviewed for calculating c_f: for elements with sharp edges. The first method uses BS EN 1991-1-4 Clause 7.7 and the other BS 6399 clause 2.7. The values in table NA2 in the National Annex to BS EN 12899 are based on the slightly more conservative results achieved using BS EN 1991.

3.9 The BS EN 1991-1-4 Clause 7.7 method calculates $c_f = c_{f,o} \cdot \psi_\lambda$ with $c_{f,o} = 2$ for all cases. The end effect is calculated from clause 7.13, which uses table 7.16 and figure 7.36 to obtain the factor. Section 1 is valid for signs as all will be less than 15m long. The rule in this case as $\lambda = 2\ l/b$, which is where this method is more conservative than the National Annex. Fig 7.36 gives a reduction factor based on the aspect ratio with a solidity ratio of 1 being valid for signboards.

3.10 The National Annex to BS EN 1991-1-4 replaces table 7.16 of the main standard with table NA.6. This results in a change of formula for the slenderness ratio to: $\lambda = l/b$.

3.11 The limit values are as follows:

Code	l/b	λ	ψ_λ	c_f
EN 1991-1-4	1	2	0.63	1.26
NA to BS EN 1991-1-4	1	1	0.6	1.2
EN 1991-1-4	30	60	0.9	1.8
NA to BS EN 1991-1-4	60	60	0.9	1.8

Using this method, no signs will have a c_f close to 1.8.

3.12 It is important to note here that the reference height used in the design of elements is to the top of the element and not the centre of the area as it is for signs. This does not directly affect the calculations for c_f but may be relevant when considering this method for use on signs.

Design wind action to BS EN 1991-1-4

3.13 The examples in Appendix C show the detailed method for calculating wind action to BS EN 1991-1-4 and its UK National Annex. The examples use the shortened formula NA.3a where orography is not significant and the factor for $c_e(z)$ is taken from figure NA.7 for a known distance from the shoreline. Where orography is significant formula NA.4a/NA.4b should be used.

3.14 The wind speeds are defined by the wind map Figure NA.1 in the BS EN 1991-1-4 National Annex. The height to the centroid is defined in BS EN 1991-1-4 fig 7.21. This is used in equation 4.8 in clause 4.5, which is then used in clause 5.3 equation 5.3. The wind force is therefore defined using the height to the centre of the sign. Note that equation 4.8 is replaced by equation NA.3 in clause NA 2.17 in the National Annex.

3.15 BS EN 1991-1-4 equation 5.3 clause 5.3 uses a structure factor $c_s c_d$ which is calculated in equation 6.1 in Clause 6.3.1. This factor is not used in the calculations as it is considered insignificant for traffic signs. The National Annex to BS EN 1991-1-4 clause NA 2.20 gives guidance on the calculation of $c_s c_d$, which for most signs will be near 1.0.

3.16 The examples do not include the orography factor calculated to clause NA 2.9 and Annex A3. This is rarely significant but should be included at the calculation of $q_p(z)$. For areas with potential for wind funnelling refer to section 6 for guidance.

Wind Load values from the National Annex to BS EN 12899-1:2007

3.17 The wind values given in BS EN 12899-1 Table NA.2 are based on calculations using both BS EN 1991-1-4 and its National Annex, and taking the more conservative result. The latter method has since been confirmed as the most appropriate for the UK. The height bands were reviewed to get the most efficient wind value for each region.

3.18 The wind values assume category II terrain roughness. Terrain categories are defined in BS EN 1991-1-4:2005, clause 4.3.2, and category II and higher are the predominate terrain roughness parameters in the country. The wind action values would be excessively conservative if they all were based on the higher category I or 0. The values given cover the majority of situations with limitations of: maximum altitude; wind velocity grouping, etc. If a particular site falls in terrain category I or 0 then detailed design will be required. Exposure factors are related to the height, the smaller the height the greater the effect of the terrain roughness (terrain category). The relationship is not one that can be simplified into a general rule.

3.19 The reference heights for the wind values in the National Annex are from ground level to the top of the sign, rather than to the centroid. If the height to the centroid of the sign(s) is greater than ¾H, then the limiting overall heights 4 m and 7 m in the table should be reduced to 3 m and 5.25 m respectively.

3.20 The example calculations are based on the wind load values in the National Annex to BS EN 12899-1:2007. The values for c_f in the lookup table in the National Annex are based upon the EN 1991-1-4 procedure for thin structural elements, which is an improvement on the conservative value of 1.8 recommended for 'signboards'. The method used to calculate c_f is in accordance with BS EN 1991-1-4 which is slightly more conservative than the method in its National Annex. In the example calculations we have not shown interpolation from the table for c_f though it would be acceptable.

3.21 The table NA.2 gives a maximum sign height of 7 m, which is rarely exceeded in practice, however for the higher signs the designer should use the more rigorous method. Sign posts taller than 7 metres will require certification as Category 0 structures in accordance with CG 300, and those higher than 9 m to Category 1.

3.22 The Wind Load values in Table NA.2 are subject to the following caveats:

- The values are only valid up to an altitude of 250 m.
- The values are only relevant where there is no local funnelling effect, or significant topographical features such as cliffs or escarpments (referred to as *orography* in clause NA 2.9 and Annex A.3 of BS EN 1991-1-4). Refer also to section 6.

4. BACKGROUND INFORMATION ON SUPPORT DESIGN

4.1 The Ultimate Limit State (ULS) effects are a combination of bending moment, shear and occasionally torsion. For the support design, these are generally all at maximum at the top of the connection to the foundation. At the Serviceability Limit State (SLS) the structure will not fail structurally but will be at the limit of performance requirements, such as acceptable deflection. For the SLS check a lower wind pressure is used and the partial action (safety) factor is 1.0.

4.2 The design effects are calculated from simple analysis of loads on a cantilever including the load factors.

4.3 The designer should have access to the characteristic properties of the proposed supports. These can be developed from first principles by calculating the section area, second moment of area, etc., or from the manufacturer's published properties for individual products. The characteristic property is defined as that which 95% of the products will achieve (not the average). This is calculated by statistical analysis of test results when carried out. These values are reduced by a factor γ_m (given in Table 7 of BS EN 12899:2007) to allow for the variability of the material. The selection of the support is based on the simple criterion that the capacity of the post must be greater than the load effect.

4.4 The example calculations take no account of wind pressure on the supports themselves, i.e. on the area below the sign plate that is exposed to wind. It has been shown that this has about 2% effect on the overall actions based on a height to the centroid of the sign, which is conservative. This is considered insignificant. If the designer wishes to include a value to allow for this, it is suggested that use of γ_{f3}=1.05 would be adequate.

4.5 Torsion may be significant where the sign plate is positioned eccentrically on a single post or where the sign is part shielded by an obstruction that eliminates wind action on part of it, or when a point load of 500 N (class PL3) on the extremity of the sign is considered. The use of an eccentricity of 0.25 × the width of the sign (recommended in BS EN 1991-1-4 clause 7.4.3 (2)) is considered excessive. It is suggested that the maximum torsion is achieved with the maximum wind action on the area of sign plate one side of the post, and that the associated bending be calculated using wind on only this area and not the whole sign. Where torsion due to wind action is significant the support should be checked for the combined effect of torsion and bending.

4.6 The shear capacity check is given but in practice it is likely to be significant only for large signs at low mounting heights supported on passively safe posts.

4.7 The deflection is checked at the top of the sign where it is greatest, not at the centroid. The limit on deflection may not be suitable for non-standard signs such as VMS and those with moving components. In this situation refer to the manufacturer for specific deflection limits.

5. FOUNDATION DESIGN

Design of spread foundations to EN 1990 and EN 1997

5.1 The methodology in the following guidance has been updated for this edition assuming the following:

- The Sign is no more than 4 m height above ground to the top of the sign,
- The sign face and its supports are located symmetrically above the centre of the foundation, so that there is no significant torsional loading,
- The ground is either flat ground or rising to no closer than 3 m to the edge of base, and
- The water table at or below founding level.

In other cases, the methodology will still be relevant, but consideration should be given as to whether the assumptions made are appropriate and that the wind and other actions are within the range for the approaches described.

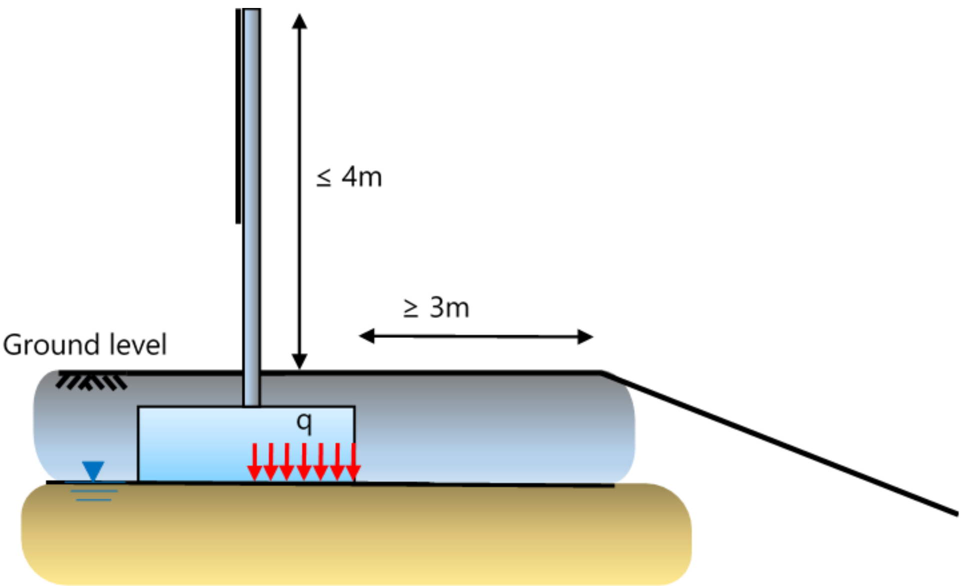


Figure 5.1 – Layout of the sign and foundation referred to in this section

5.2 The design of spread foundations requires checks for stability against sliding and bearing capacity. Design requirements are presented in EN 1997 and actions in EN 1990. BS 8004 also provides non-contradictory, complementary information for use with EN 1997. The UK national annex to EN 1997 specifies 'material factors', i.e. partial factors applied to 'characteristic' soil strength to obtain the 'design' ground resistance while the national annex to EN 1990 specifies partial factors on actions to obtain the 'design' action. A design is acceptable if the 'design' sliding or bearing resistance is greater than the 'design' action. Strengths and forces with the suffix 'k' are 'characteristic', i.e. unfactored, whereas those with the suffix 'd' are factored, i.e. 'design' values. The pressure distribution is shown as uniform across the portion of the foundation upon which it is taken to act in Figures 5.1 to 5.3 and in Appendix C Example 2. This simplification is in keeping with the sample analytical method in Annex D of EN 1997-1, but analysis using the triangular distribution that occurs in practice is also valid and is preferable for larger structures or when the reaction (discussed in 5.12 below) is not within the middle two-thirds of the base.

5.3 EN 1990 and 1997 define the following ultimate limit states:

STR: internal failure or excessive deformation of the structure or structural elements … in which the strength of structural materials is significant in providing resistance

GEO: failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance

5.4 The STR limit state is failure of the structure. The GEO limit state includes sliding and bearing failures as defined in previous UK codes such as CP2, BS 8002 and BS 8004. Passive resistance of the soil against the side of the foundation should be generally be ignored in sliding and always in bearing, whereas weight of soil above the foundation may be included in calculation of design bearing pressure.

5.5 Design Approach 1 is specified in the UK national annex to EN 1997 for checks on STR and GEO limit states. In Design Approach 1, two Combinations 1 and 2 of material and action factors are checked for both STR and GEO. These are also referred to as A1 '+' M1 '+' R1 and A2 '+' M2 '+' R1 respectively. EN 1997 states *If it is obvious that one of the two combinations governs the design, calculations for the other combination need not be carried out. However, different combinations may be critical to different aspects of the same design.* The GEO limit state is often governed by Combination 2 and the STR limit state by Combination 1, but where live load is a significant proportion of the total, as it is with signs, Combination 1 may govern and should therefore be checked as well as Combination 2. Note that where the permanent load is treated as stabilising, see the partial factors in Table 1, Combination 1 will govern foundation load eccentricity.

5.6 EN 1990 and 1997 also define an EQU limit state:

EQU: loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance

In the context of spread foundations, the EQU limit state is equivalent to an overturning check as defined in previous UK codes and is intended to check overturning about the edge of the base without failure of the ground. BS EN 1997-1 notes that *In geotechnical design, EQU verification will be limited to rare cases, such as a rigid foundation bearing on rock.* While the partial load factors for EQU are slightly more onerous than those for GEO, see Table 5.1, if the requirement to limit eccentricity of vertical load in bearing to the 'middle two thirds' is met, see below, EQU will be satisfied by inspection.

5.7 The national annex to EN 1997 includes partial factors on soil strength for the EQU limit state. However, these apply in the case of finely balanced structures and are not therefore relevant to sign spread foundation design. As noted in 'Concise Eurocodes: Geotechnical design, BS EN 1997-1: Eurocode 7, Part 1' (BSI, 2011) sliding and bearing checks for the EQU limit state are therefore not required in addition to GEO sliding and bearing checks.

5.8 The national annex to EN 1997 specifies that the Set A, Set B and Set C partial factors on actions given in the national annex to BS EN 1990 are to be applied in the EQU, STR/GEO Combination 1 and STR/GEO Combination 2 checks respectively, see Table 5.1.

5.9 For the range of assumed design bearing resistances in Table 5.2 below a settlement check is considered not to be required for typical spread foundations supporting only sign loading. Thus inclusion of an additional 'allowable' resistance to limit settlement, e.g. the BS 8004 'presumed bearing resistance' of one half to one third characteristic ultimate resistance, is not necessary.

5.10 In summary, bearing and sliding (GEO limit state) will often be governed by Design Approach 1 Combination 2 soil partial factors with corresponding Set C factors on actions but Combination 1 with Set B factors on actions may govern and should also be checked. Combination 1 treating permanent load as stabilising will govern foundation load eccentricity. The overturning check (EQU limit state) will be satisfied by inspection if bearing load eccentricity requirements are met. EQU may govern where bearing is not relevant, such as a foundation on strong rock.

5.11 The factors for building structures are recommended because they are similar to those adopted in historic sign designs and are less onerous than the alternative for bridges. The following table shows the relevant factors for Eurocode method for building structures based on the current UK national annexes to BS EN 1990 and BS EN 1997-1.

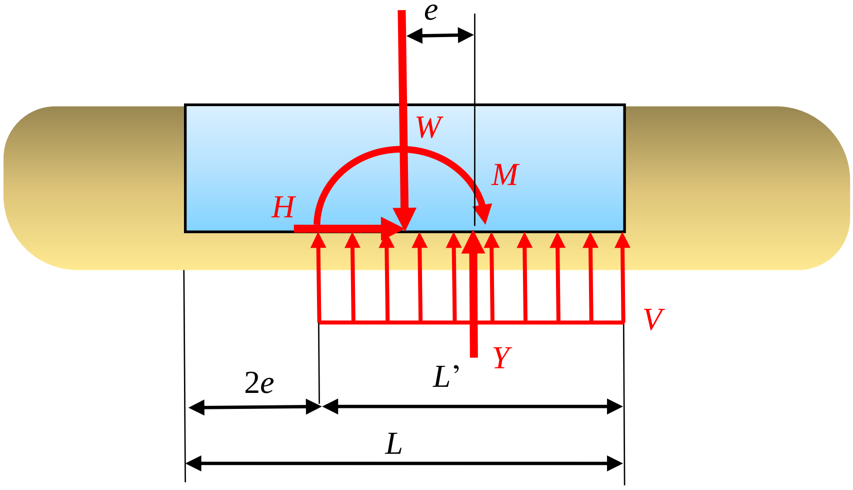
Load or resistance		**EN 1990 – GEO Design Approach 1 – Combination 1 'DA1C1' (Set B load factors)**		**EN 1990 – GEO Design Approach 1 – Combination 2 'DA1C2' (Set C load factors)**		**EN 1990 – EQU Limit State [a] (Set A load factors)**	
Variable loads, γ_Q		Unfavourable	Favourable	Unfavourable	Favourable	Unfavourable	Favourable
		1.5	0	1.3	0	1.5	0
Permanent loads, $\gamma_{G,sup}$ and $\gamma_{G,inf}$		Destabilising	Stabilising	Destabilising	Stabilising	Destabilising	Stabilising
		1.35	1.0	1.0	1.0	1.1	0.9
Soil param-eters	$\gamma_{\phi'}$	1.0		1.25		n/a	
	γ_{cu}	1.0		1.4		n/a	

[a] Overturning about edge of foundation, satisfied by inspection if eccentricity of load in the GEO bearing checks is acceptable.

Table 5.1: EN 1990 and EN 1997 factors for the design of sign bases

5.12 Bearing may be checked using the method presented in Annex D of EN 1997 in which an average design bearing resistance is derived over a reduced bearing area. EN 1997 requires 'special precautions' if the eccentricity e exceeds one third of the width of a rectangular footing because bearing resistance reduces rapidly at greater eccentricities. When checking bearing, eccentricity must therefore not exceed one third the width of the base, i.e. the resultant must remain within the 'middle *two* thirds'. (This is less onerous than the 'middle third' requirement to maintain positive contact pressure across the whole width of the foundation.)

5.13 The most efficient base is achieved with minimum depth and breadth w (parallel to the sign face), and increasing the length L (perpendicular to the sign face) to achieve stability.



$e = M / W$ $\quad$ $V = W / (L - 2e)$ $\quad$ e not to exceed $L/3$

L = Width of foundation
M = Overturning action
Y = Resultant soil reaction
W = Breadth of foundation (into page, parallel to sign face)
e = Eccentricity of reaction
V = Bearing pressure
W = Vertical action

Figure 5.2: Bearing pressure diagram for spread foundation

5.14 The strength of soils is usually based on the assumption of poor soils, as experience has shown that it is unusual for a ground investigation to be carried out prior to installing signs. A DA1C2 design bearing resistance of 100 kN/m^2 (135 kN/m^2 for DA1C1, see Table 5.2) is recommended as a minimum for bearing checks to EN 1997 on a typical highway embankment or cutting under wind loading conditions. (As noted above, 'material factors' are applied to 'characteristic' soil strength in EN 1997 to obtain a 'design' bearing resistance. A design is acceptable if the 'design' bearing resistance is greater than the 'design' bearing pressure which includes partial factors on the actions.)

5.15 In other locations, including green-field situations, design to EN 1997 using soil parameters interpreted from site investigation data may be adopted, in which case both DA1C1 and DA1C2 checks should be carried out for eccentricity, bearing and sliding. The bearing resistance of spread foundations is affected not only by soil and groundwater conditions and eccentricity of vertical load but is also sensitive to magnitude of horizontal load compared to soil strength and to load inclination, i.e. the ratio of horizontal to vertical load. Bearing checks to EN 1997 should therefore consider the effect of horizontal load, applying the inclination factors presented in Annex D of EN 1997-1.

5.16 Alternatively, if soil parameters interpreted from site investigation are not available, assumed design bearing resistances may be adopted as shown in Table 5.2 below, subject to the design, field inspection and testing requirements in Table 5.2 and Figure 5.3. Note that the worked example contained in this guide is based on an assumed design bearing resistance.

Soil type	Assumed design bearing resistance		Required strength and corresponding soil description on ground investigation logs. N.B. strength to be verified by field testing using Dynamic Cone Penetrometer (DCP) testing, see Figure 5.3.	
	DA1C2	**DA1C1**	**Cohesive soil**	**Granular soil**
Poor	100 kN/m^2	135 kN/m^2	Shear strength 30 kPa, soft to firm or low to medium strength clay.	Angle of friction φ' 30°, SPT N of 5, loose sand or gravel.
Average	150 kN/m^2	205 kN/m^2	Shear strength 45 kPa, firm or medium strength clay.	Angle of friction φ' 34°, SPT N of 10, loose to medium dense sand or gravel.
Good	200 kN/m^2	275 kN/m^2	Shear strength 60 kPa, firm to stiff or medium to high strength clay.	Angle of friction φ' 37°, SPT N of 15, medium dense sand or gravel.
The above required (rounded) strengths correspond to 0.75 m founding depth. For a given bearing resistance the required cohesive shear strength remains similar at 0.5 m and 1.0 m depths and reduces slightly with increasing depth thereafter. The required φ' is approximately 3° higher at 0.5 m depth and 3° lower at 1.0 m depth than the values shown for 0.75 m depth, with corresponding variations in the required SPT N. The required DCP mm/blow given in Figure 5.3 brackets both soil types. The required mm/blow is generally governed by cohesive soil at founding depths of 0.75 m or greater. Therefore if it can be confirmed that the bearing stratum is granular soil, design to EN 1997 at depths of 0.75 m or greater using soil parameters interpreted from site investigation may give higher design bearing resistances than those shown above. Corresponding log descriptions are based on required strengths and classification in BS 5930:2015: Low clay shear strength = 20-40 kPa, medium strength = 40-75 kPa and high strength = 75-150 kPa; Uncorrected SPT N of loose sand or gravel = 4-10, medium dense SPT N = 10-30 and dense SPT N = 30-50.				

Table 5.2: EN 1997 Assumed design bearing resistances based on soil type for signs meeting the requirements of Table 5.3

5.17 As noted above, bearing resistance is affected by horizontal load. Use of the above assumed design resistances is therefore subject to the requirements in Table 5.3 which include limits on magnitude of horizontal load and load inclination. These limits have been set such that most small signs subject to wind load should satisfy them. The assumed design resistances above also ignore the benefit of any shape factor and are therefore valid for any foundation breadth w into the page, see Figure 5.2. If the If the requirements cannot be met, specialist advice should be sought.

5.18 In applying the above assumed design bearing resistances the load eccentricity should first be checked. As noted above, this will be governed by the DA1C1 check assuming the foundation weight to be stabilising. Note that the limits on horizontal load in Table 5.3 related to load inclination will also result in the sliding checks being satisfied: $H_d \leq 0.36(L' \cdot \mathrm{w} \cdot c_{u;d})$ is by inspection more onerous than the sliding resistance requirement $H_d \leq L \cdot w \cdot c_{u;d}$ given in 6.5.3(11) of BS EN 1997-1. Also, $\mathrm{H_d} / \mathrm{W_d} \leq 0.15$ is more onerous than the sliding requirement $H_d \leq 0.4W_d$ given in 6.5.3(12) of BS EN 1997-1. Moreover, $H_d / W_d \leq 0.15$ corresponds to characteristic friction angles δ of 9° in DA1C1 and 11° in DA1C2 (where $R_d = V'_d \cdot \tan\delta_d$ given in 6.5.3(8) of BS EN 1997-1), which will be satisfied for all ground provided the foundation is in contact with it, i.e. absence of low-friction layers such as polythene below the foundation.

A General requirements	A1	Size: Less than 4 m height above ground to top of sign board with flat or rising ground to a minimum distance of 3 m from edge
	A2	Depth: Founding depth minimum 0.5 m below ground
B Bearing check and horizontal load requirements (Horizontal load requirements will satisfy sliding check)		Select assumed design bearing resistance from Table 5.2 based on available information (To be confirmed by field testing, see below)
	B1	Eccentricity and bearing width: Check $e \leq L/3$ treating foundation weight as stabilising, <u>and</u> bearing width $L' \geq 0.5\,\mathrm{m}$, see Figure 5.2 for definition of e and L' (DA1C1 with permanent load considered as stabilising, $\gamma_{G,\mathrm{inf}} = 1.0$, will govern)
	B2	Bearing: Check bearing pressure less than assumed design bearing resistance (DA1C2 likely to govern but also check DA1C1)
	B3	Horizontal load [2] (DA1C2 likely to govern): Cohesive soil: $H_d/(L' \cdot w \cdot c_{u;d}) \leq 0.36$, see Figure 5.2 for definition of L', noting w = foundation breadth, where $c_{u;d}$ is 18, 29, 40 kPa for assumed DA1C2 design bearing resistances of 100, 150 and 200 kPa respectively and $c_{u;d}$ is 25, 41, 56 kPa for assumed DA1C1 design bearing resistances of 135, 205 and 275 kPa respectively <u>and</u> Granular soil [3]: $H_d / W_d \leq 0.15$
C Field inspection and testing requirements [1]	C1	Soil strength: Perform testing in each corner of excavation formation to confirm assumed design bearing resistance as shown in Figure 5.3 below
	C2	Soil weight: Confirm that soil above founding level is not peat (Satisfies unit weight ≥ 16 kN/m^3 used to derive design resistances)
	C3	Water table: Confirm water table is at or below founding level
D Contingency measures if field tests not met	D1	If inspection and/or test criteria not met in bearing layer, excavate and replace with compacted Type 1 material to SHW Cl 803 to depth D and width D outside base as shown in Figure 5.3 below If test criteria not met in ground below bearing layer, excavate and replace to below failed test depth and to distance D' from all four sides of footing with compacted Type 1 material to SHW Cl 803 as shown in Figure 5.3 below, or seek specialist advice

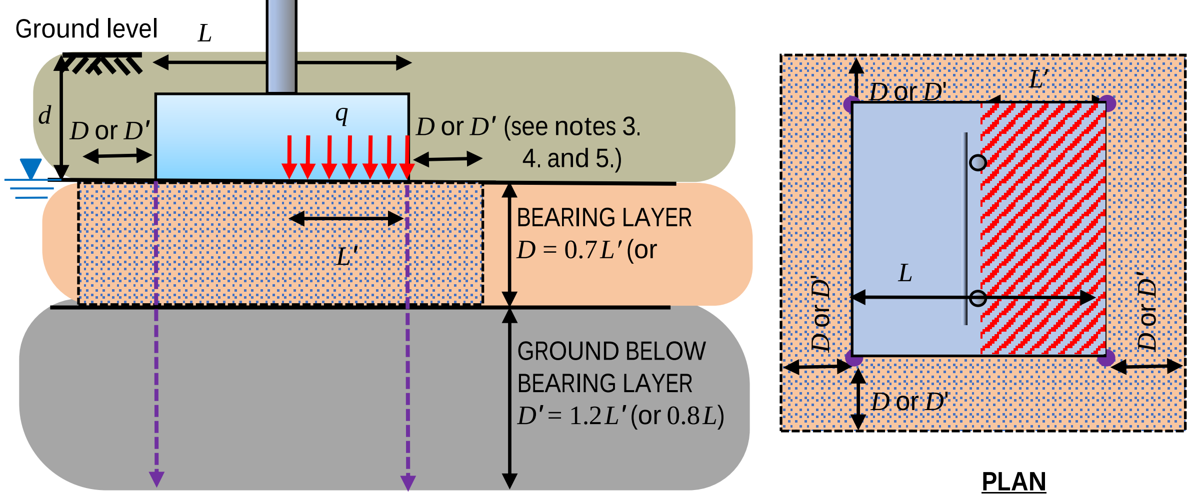
[1] Not required for lowest, 100 kN/m^2 assumed design bearing resistance if sign is founded in a highway embankment or cutting designed to DMRB standards.
[2] The requirements on horizontal load are included in order to limit load inclination, which affects bearing resistance. If these are met, sliding checks to EN 1997 will also be satisfied. The first requirement relates to cohesive ground and the second to granular ground, both being required when using assumed resistances as the ground type is unknown. The shear strengths given are less than the (rounded) values given in Table 5.2 as they either include partial factors or correspond to 1.0 m depth or deeper.
[3] $H_d / W_d \leq 0.15$ check only required for DA1C2, which governs drained bearing for the cases considered.

Table 5.3: Requirements for use of design bearing resistances given in Table 5.2

Dimension L' is to be determined from design calculation, see figure 5.2 for explanation.
If this is not known, use L. (This will result in increased depth of testing/ replacement).
N.B. L and L' are measured perpendicular to sign face.

<u>**Test and replacement procedure**</u>

1. Excavate and prepare formation.
2. Inspect to confirm absence of groundwater or peat above founding level.
3. Carry out Dynamic cone penetrometer (DCP) tests in all four corners of formation to depth $D + D'$ as shown, where $D = 0.7\ L'$ (or $0.5\ L$, if L' not known) and $D' = 1.2\ L'$ (or $0.8\ L$)
4. **If groundwater or peat present above founding level, or any test result in bearing layer fails** to meet criteria below, excavate and replace with compacted Type 1 material to SHW Cl 803 to **depth D and distance D** from all four sides of footing as shown.
5. **If any test in ground below bearing layer fails** to meet criteria below, excavate and replace to **below failed test depth and to distance D'** from all four sides of footing as shown with compacted Type 1 material to SHW Cl 803, or seek specialist advice.



<u>**SECTION**</u>

<u>**PLAN**</u>

Dynamic Cone Penetrometer (DCP) and test method is defined in
Clauses 6.33 to 6.39 of CS 229 Data for pavement assessment (formerly HD29/08)

Founding depth d, below ground (m)	**Design bearing resistance DA1C2 (kPa)**	**BEARING LAYER**	**GROUND BELOW BEARING LAYER**
		mm/blow to be equal to or less than	
0.5m	100	55	135
0.5m	150	25	85
0.5m	200	15	60
0.75m	100	70	145
0.75m	150	45	90
0.75m	200	25	65
1.0m or deeper	100	70	160
1.0m or deeper	150	45	95
1.0m or deeper	200	30	65

N.B. mm/blow to be achieved through all of test depth, calculated over 100mm depth intervals.

Figure 5.3: Soil testing and replacement requirements

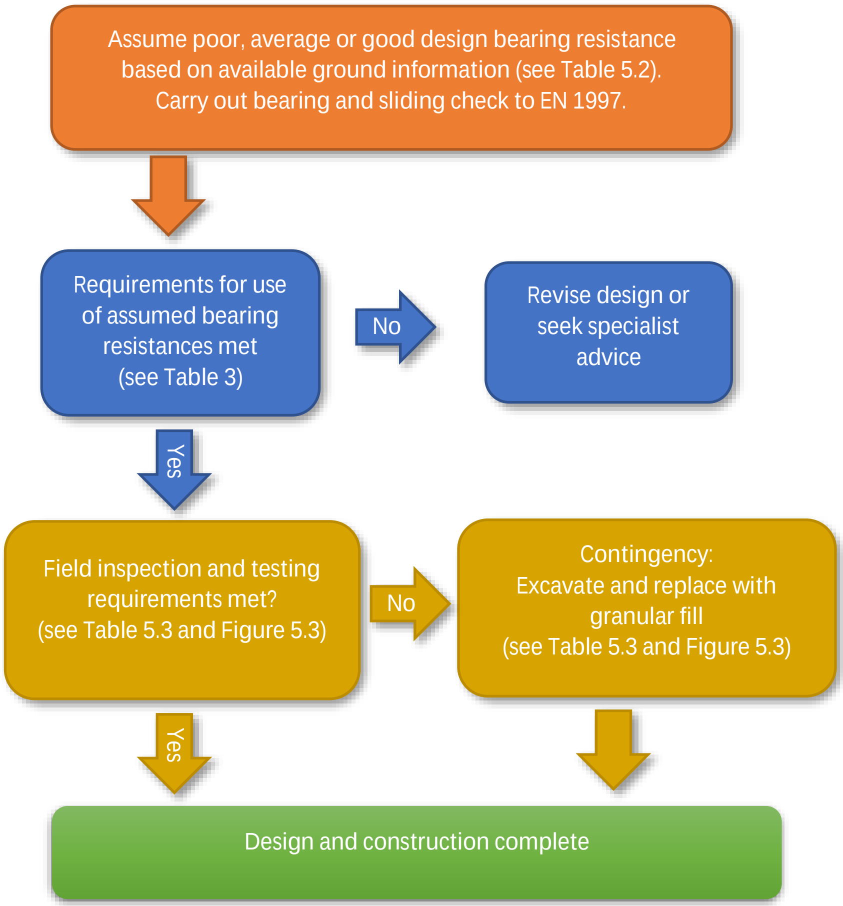


Figure 5.4: Spread foundation design using assumed design bearing resistances

Design of planted foundations to CD 354 and PD 6547

5.19 Planted foundations are based upon the method detailed in PD 6547 (*Guidance on the use of BS EN 40-3*), which relates to lighting column foundations. Research commissioned by Highways Agency (now Highways England) enabled this method to be extended to traffic sign structures in the former standard BD 94/07. This same method persisted in the replacement standards, BD 94/17 and the initial revision 0 of CD 354 of December 2019. However, revision 1 of CD 354 of March 2020 has modified the method for signs so that it no longer aligns with that for lighting columns. The result of the change is to disallow the inclusion of the diameter of the surrounding backfill material when calculating the ground resistance moment, relying purely on the interaction of the sign post with the surrounding soil. It is understood that this change related to the need to eliminate poorly constructed foundations, rather than any problem or deficiency in those sign bases correctly installed to these previous standards. The previous method is therefore still recommended where permitted for the road in question, in view of the wealth of experience gained of planted sign foundations during the last 14 years. It is referred to as the PD 6547 method, to reference it to a current standard and to distinguish it from strict compliance with CD 354. Example calculations using the methods of both standards are given in Appendix C sections 1.5 to 1.7.

5.20 For either standard, the design uses unfactored actions and resistance and applies a defined factor of safety of 1.25 for overturning of the foundation to the destabilising action. The stability of this type of foundation relies on the passive resistance of the soil surrounding the foundation which must be well compacted. For this reason, the use of planted foundations on a slope or near ditches or excavations require special consideration as discussed below. Regardless of the orientation of the sign face, the effect of a slope should be considered, but it becomes increasingly important the closer the sign plate is to perpendicular to the line of greatest slope. This design method is only appropriate for foundations where the depth is significant in relation to the diameter (twice the diameter would be preferable, but 1.5 times would generally be adequate for small signs on level ground) in order to mobilise the passive resistance of the soil. Minimum planting depth requirements are given in BS EN 40-2 Table 7. The thickness of any soft fill material above the foundation should not be considered part of the planting depth. CD 354 clause 12.7 recommends using the middle column of planting depth values from Table 7 for traffic signs and signals, but allows a reduced depth of 600 mm for signs under 2.0 m total height provided that the requirements of clause 12.7.2 are met. Precautions should be taken when constructing planted foundations to ensure that they do not damage any statutory undertaker's service.

Planted foundations on a slope

5.21 Planted foundations on a slope can be designed using a simple conservative approach based on the assumption that the top layers of soil are not effective. Refer to Figure 5.5. Two alternative methods are proposed to define a *Notional Ground Level* for calculating actions and resistance. The calculations are otherwise identical to those for a normal planted foundation, but using a greater height of post and a reduced planting depth. The first method is based on providing a minimum horizontal distance of 3 m to the nearest edge of the slope and the second method on ignoring a proportion of the total foundation depth, the percentage varying with the slope angle. The slope angle should be the highest value on the downhill side of the foundation within 3 m of the post. The slope of the ground above the position of the sign is not significant and can be ignored.

5.22 The methods proposed are considered adequate under normal circumstances for the worst case of a sign face perpendicular to the line of greatest slope. In the majority of cases, the sign face is parallel to this line, so the wind action is along the contour of the slope. The sides of the foundation resisting the sign overturning are therefore supported by the full depth of soil continuing indefinitely. In these circumstances, it is suggested that, provided the ground is of sound material, the above correction can be reduced to 50% of the amount that would otherwise apply.

5.23 When planting a foundation on a slope there is an additional risk of causing temporary or long-term instability to the slope. In cohesive material the excavations may introduce a water path into the slope, which could lead to failure.

5.24 Planted foundations for signs on slopes will have significantly deeper planting depths and greater access problems. A planted foundation in these cases may not be achievable and will pose a greater risk to site operations, so it may be more appropriate to provide a standard spread foundation. Another alternative would be to auger a concrete base and provide a socket or bolted connection for the post. Careful consideration of the need for reinforcement is required in this case.

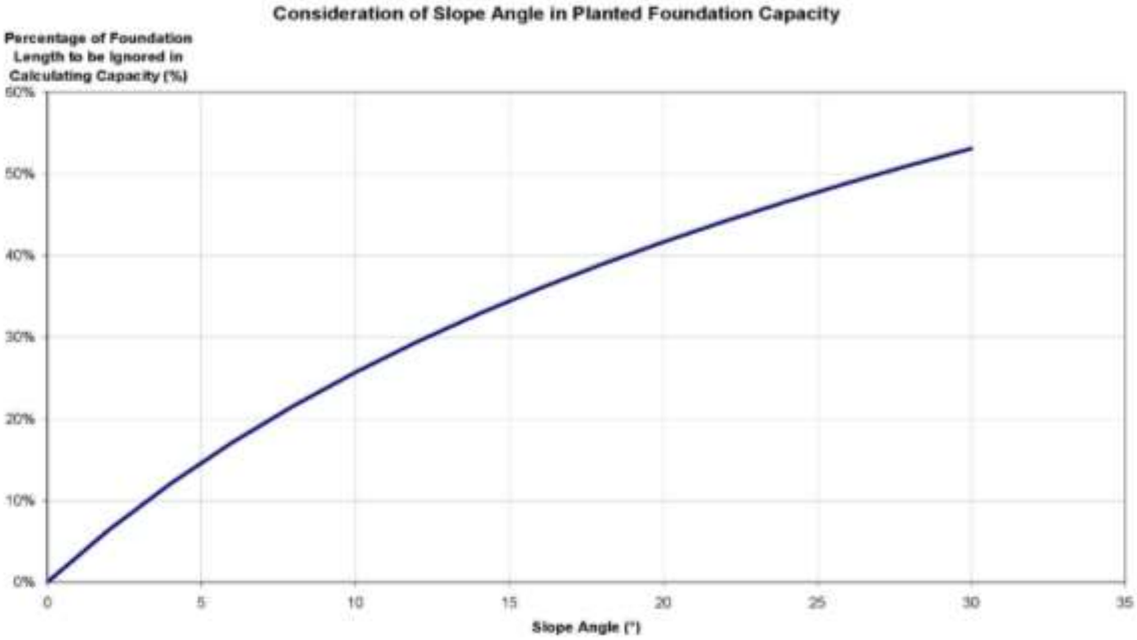


Figure 5.4 F_{slope} *for determining* h_b *using Method 2*

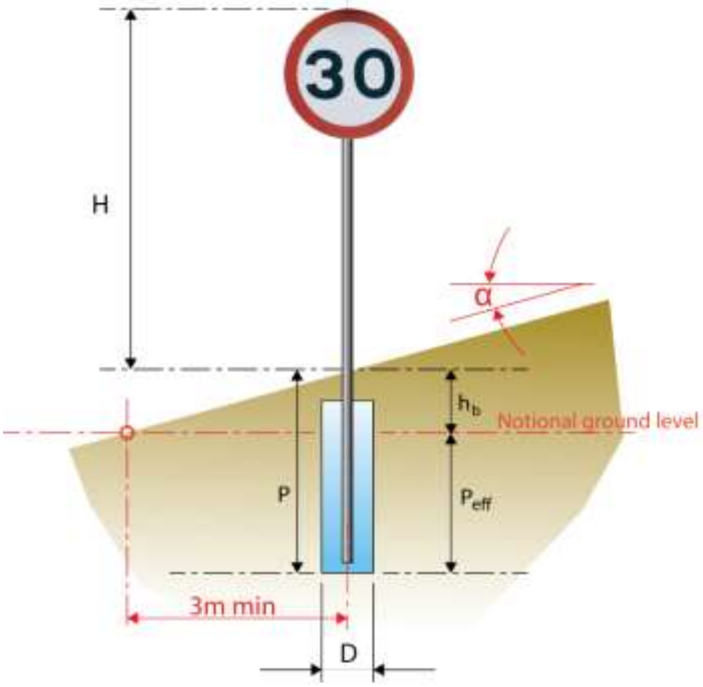


Figure 5.5
Planted foundation on a slope

Angle of slope	α
Mounting height above ground	h_m
Width of sign face	l
Height of sign face	b
Total height = $h_m + b$	H
Height to centroid of sign area = $h_m + b/2$	z
Effective depth of post buried above foundation	h_b
Minimum diameter of foundation (for CD 354, D is the diameter of the post)	D

Calculation of revised h_b for sign on slope:

Method 1	h_b = 3 metres × tan α	
Method 2	$h_b = F_{slope} \cdot P$	where F_{slope} is read from Figure 5.4

For either method:

Foundation support planted in plain concrete:
minimum diameter of foundation in the ground. D
planting depth of foundation. P

Effective planting depth $P_{eff} = P - h_b$

P_{eff} to be used in calculations in place of P. See Appendix C Example 1 section 1.6 and 1.7

5.25 The parameters can be used for the design of planted foundations as illustrated in Appendix C example 1 section 1.5. Note that P_{eff} should be used in place of P in all parts of the calculation, including the minimum planting depth from EN 40 and the depth to diameter ratio.

5.26 Example 1 in Appendix C (section 1.5) has been reworked for a 15° slope using both methods described above (sections 1.6 and 1.7). Using PD 6547, the total planting depth increases from 0.8 m to 1.6 m using Method 1, or to 1.25 m using Method 2. Method 2 is less conservative but involves more iteration. As the sign face as shown is parallel to the line of greatest slope, in sound soil the value of h_b could be reduced by 50%, resulting in a total planting depth of 1.05 m (using Method 2). The equivalent results using CD 354 can be seen in the second part of each of sections 1.6 and 1.7.

Foundation Reinforcement

5.27 There can be structural or contractual reasons why a foundation may require reinforcement. For trunk roads and other situations where DMRB standard CD 354 is to be followed, reinforcement is made a requirement for spread foundations to BS EN 1997-1, unless a departure is sought. For planted foundations, CD 354 indicates that reinforcement is not required, and also provides for precast concrete foundations. Clause 12.2 notes that *Unreinforced concrete spread foundations rely on the tensile strength of concrete and are unlikely to meet the requirements for durability.* For situations where CD 354 does not apply, it nevertheless provides useful guidance, but clients may accept unreinforced foundations where they can be shown to be satisfactory structurally, and an example of the necessary calculation is given in section 2.5.11 of Appendix C. In practice many spread foundations are unreinforced, particularly for smaller signs.

5.28 The previous (2010) edition of this Guide advised:

> *Reinforcement may be omitted for smaller signs and squarer bases where it can be shown that the tension due to bending in the concrete is not significant. In general foundations for smaller CHS posts (89 mm diameter or less) do not need reinforcement.*

The authors are not aware of any case where following this advice has led to a failure.

5.29 Where reinforcement is provided, some designers will consider it advisable to add additional steel to control the widths of cracks that might occur within the slab. This is for reasons of durability, to avoid moisture penetration that could lead to corrosion of the main reinforcement. For some structures it would also avoid loss of shear capacity or be for aesthetic reasons, but neither of these applies to a buried slab with minimal vertical loads. Only cracks caused by thermal action and shrinkage are of concern, as those caused by bending will recover once the wind abates, so are not a durability issue. For a design life of typically 25 years for a traffic sign, it is unlikely that the reinforcement will corrode enough in this time for a significant loss of strength to occur. Any major cracks on the upper surface can be sealed if they are a durability concern.

5.30 Although it would be unusual for a sign of the size envisaged in the examples in this Guide (4 m maximum height) to have crack control steel in its foundation, this Guide nevertheless provides an example of the calculations needed to design this type of additional reinforcement.

6. WIND FUNNELLING AND TOPOGRAPHY

6.1 At present there is no published guidance available for design in areas of wind funnelling. This section provides recommendations for wind funnelling following a study to define where funnelling occurs and to determine simple procedures to use in these situations.

6.2 Wind direction may be significantly changed by topography so it is recommended that the wind direction factor be ignored for the design of minor structures. The direction factor c_{dir} of BS EN 1991-1-4 should be set to 1.0 for the design of all minor structures.

6.3 The effect of all two-dimensional features such as embankments and escarpments should be included irrespective of the orientation of a sign mounted on it. The effect of an embankment can either be allowed for by using an orography factor or by adding the height of the embankment to the sign height for calculation of wind speed.

6.4 Funnelling may be neglected for traffic signs of height 5 m or less where they are designed to the simplified wind pressures of the National Annex to BS EN 12899-1.

6.5 The standard for minor structures, CD 354 should be clarified as follows:

- Clause 4.1: the reference to BS EN 40 for design of lighting columns should be supplemented by reference to PD 6547.
- Clause 5.4: the definition of "sites subject to significant local wind funnelling" includes sites where steep-sided valleys or cuttings are present, or where a highway runs along a steep-sided slope (i.e. funnelling around the side of a hill). It is not necessary for a valley to be narrowing for funnelling to occur, nor for the valley to be parallel to the roadway − funnelling may be caused by gullies running transversely to the road alignment.

6.6 Wind funnelling should be considered where three-dimensional topography occurs and a simple orography approach cannot be used. In the absence of detailed guidance, the size of significant features may be assessed using methods for orography in EN 1991 clause 4.3.4, A3 and Fig NA.2

6.7 In the absence of comprehensive guidance on funnelling the following interim advice is given. Where a structure is in a site subject to possible wind funnelling, the topography factor $c_o(z)$ of BS EN 1991-1-4 (factor f of BS EN 40-3-3) shall be a minimum of 1.4 at heights up to 5 m, reducing to 1.2 at heights of 10 m and above (with linear interpolation between). Where funnelling features exist in combination with other significant topography (e.g. gullies cut into a steep hillside), consideration should be given to adopting a higher topography factor of 1.65 at heights up to 5 m, reducing to 1.2 at heights of 10 m and above.

6.8 If the location is more complex, expert advice must be sought.

APPENDIX A: Table NA.2 of National Annex

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Recommended classes or values for physical performance most suitable for UK practice

Property	Recommended performance class or value			
Wind load	**EITHER** design to BS EN 1991-1-4:2005+A1:2010 using the 10-minute mean wind reference speed appropriate to the locality of the sign, taken from the national wind map (shown in BS EN 1991-1-4 UK National Annex) and adjusted for altitude			
	OR wind load values may be taken from the table below, which has been calculated using BS EN 1991-1-4. The first method is however preferable and more economical (see Note 1).			
	Location	**Sign maximum overall height H (m)** See Note 2	**Distance from the shoreline (d)**	
			d ≤ 5 km	**d > 5 km**
			Wind load value (kN m^{-2})	
	England	4.0	1.0	1.0
		7.0	1.3	1.2
	Wales	4.0	1.1	1.0
		7.0	1.3	1.2
	Northern Ireland & Isle of Man	4.0	1.3	1.2
		7.0	1.5	1.4
	Scottish mainland	4.0	1.5	1.4
		7.0	1.8	1.7
	Scottish islands	4.0	1.6	1.5
		7.0	2.0	1.8

Table NA.2 (continued)

Property	Recommended performance class
Partial safety factors	Class PAF 1, Table 6 in BS EN 12899-1:2007.
Partial material factors	According to the material used in the manufacture of the sign, see Table 7.

NOTE 1 Using BS EN 1991-1-4 will generally result in a significantly lower wind load and therefore a more economical structure than using the values above. Software is readily available to simplify the process and to provide the necessary wind speed data, but designers should check that such products are appropriate and comply with current versions of the relevant standards, as they remain responsible for the final design.

NOTE 2 The sign maximum overall height H for the wind load value is measured from ground level to the top of the sign assembly and not to the centroid of the sign. If the height to the centroid of the sign or signs is greater than ¾ H then the maximum overall heights of 4 m and 7 m in the table above should be changed to 3 m and 5.25 m, respectively, or the structural design should be to BS EN 1991-1-4.

NOTE 3 All the wind load values given in the above table apply up to a limiting altitude of 250 m above sea level (at ground level). Above this altitude, structural design of all signs should be to BS EN 1991-1-4.

NOTE 4 The wind load values in the table above are based on a wind speed return period of 25 years. Designers may choose to use a return period of 50 years when designing signs to BS EN 1991-1-4.

NOTE 5 The wind load on any solar panel or other significant additional component on the sign structure should be considered.

Table showing force coefficients c_f to be used in accordance with subclause 7.7 of BS EN 1991-1-4:2005.

Aspect ratio	1	1.6	3	5.5	7.5	13.5	20	30
Force coefficient c_f	1.26	1.3	1.35	1.4	1.5	1.6	1.7	1.8

Property	Recommended performance class		
Point loads	Signs supported by a single circular post	Class PL1, Table 10	
	Signs supported by more than one post or by a non-circular section	Class PL3, Table 10	
Dynamic snow loads	If snow blowers or snow ploughs are not regularly used	Class DSL0, Table 9	
	If snow blowers are regularly used	Class DSL1, Table 9	
	If snow ploughs are regularly used	Ploughing speed of 40 mph or less	Class DSL2, Table 9
		Ploughing speed greater than 40 mph	Class DSL4, Table 9

<table>
<tr><th colspan="5">Table NA.2 (continued)</th></tr>
<tr><td rowspan="6">Temporary deflection of sign plates and supports</td><td colspan="4">Use the appropriate temporary deflection bending class, and temporary deflection torsion class given in the table below.</td></tr>
<tr><td rowspan="2" colspan="2">Product</td><td colspan="2">Recommended performance class</td></tr>
<tr><td>Bending class</td><td>Torsion class</td></tr>
<tr><td colspan="2">Sign plate</td><td>TDB4, Table 11</td><td>n/a</td></tr>
<tr><td colspan="2">Support – not passively safe (Class 0 in BS EN 12767)</td><td>TDB4, Table 11</td><td>TDT4, Table 12</td></tr>
<tr><td colspan="2">Support – passively safe (compliant with a performance class from BS EN 12767)</td><td>TDB5, Table 11</td><td>TDT4, Table 12</td></tr>
<tr><td colspan="2">Piercing of sign face</td><td colspan="3">Class P3, Table 13</td></tr>
<tr><td colspan="2">Edging of sign plates</td><td colspan="3">Class E1, Table 14</td></tr>
<tr><td colspan="2">Corrosion protection</td><td colspan="3">Class SP1 or SP2, Table 15</td></tr>
<tr><td colspan="5">NOTE 6 The aspect ratio is the larger of l/b or b/l where b is the height and l is the width of the sign face (including any backing board and light spill screen).

NOTE 7 It is recommended that for very exposed sites, or sites subject to local funnelling effects, signs should be designed to BS EN 1991-1-4:2005.

NOTE 8 The wind load values given above are conservative. Designers may derive a wind load value for a specific location or defined area by using BS EN 1991-1-4:2005.

NOTE 9 Where the material properties or method of jointing are not known, the designer should select the highest value for partial material factor.

NOTE 10 The wind load on the sign structure is obtained by multiplying the wind pressure (the wind load value in the table above) by the force coefficient cf and the overall safety factor.

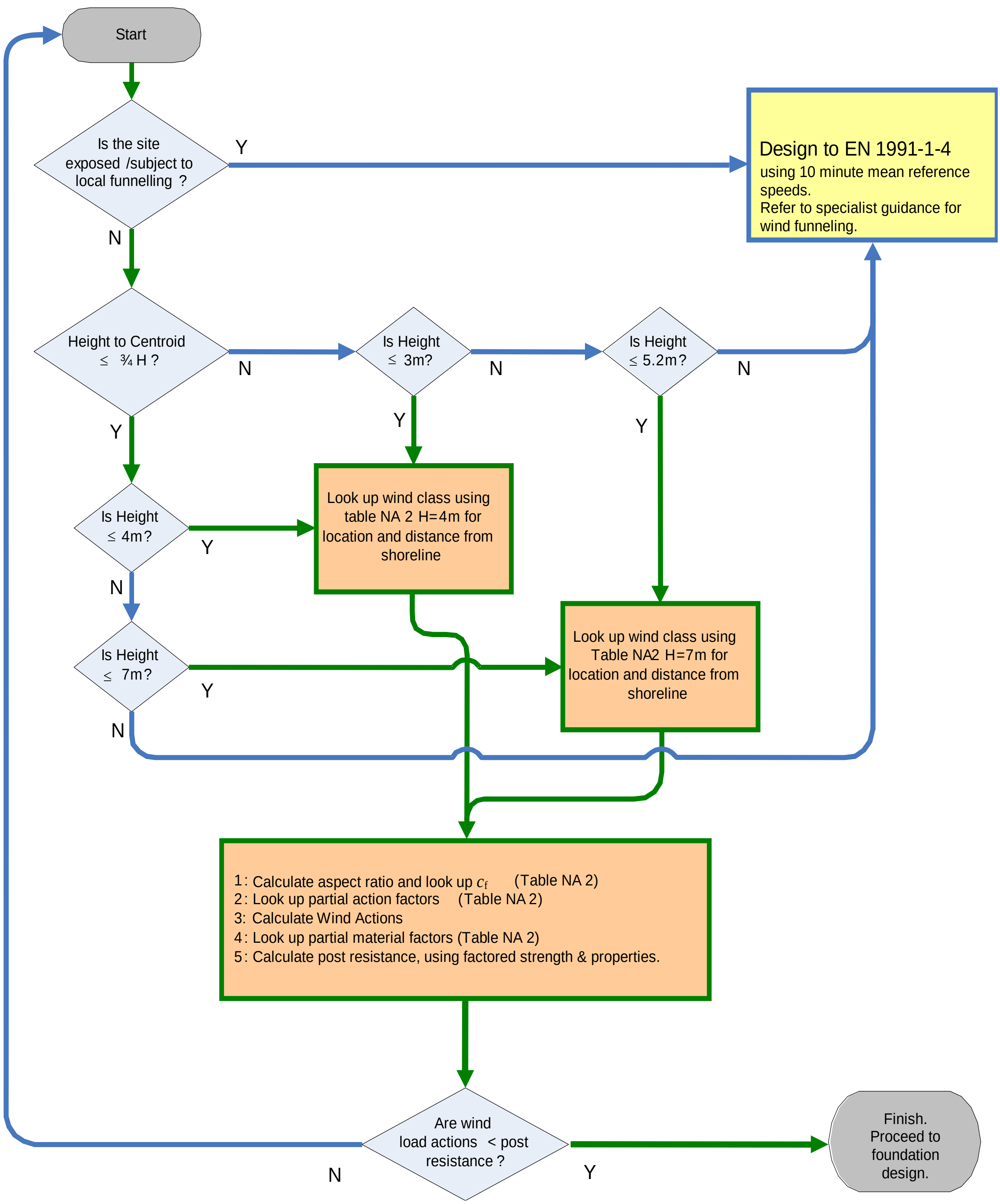
NOTE 11 The eccentricity should normally be zero and the force coefficient cf should be taken from the table above, unless calculated in accordance with Subclause 7.7 of BS EN 1991-1-4:2005.

NOTE 12 It is for the designer to decide whether to include snow loading in a design by selecting a class other than DSL0. This will usually be necessary only in locations where there is considered to be a significant problem of damage to signs during snow clearing operations.

NOTE 13 If snow blowers are correctly aligned while in use, the load on the sign should be minimal.

NOTE 14 The deflection of the sign plates should be evaluated relative to the supports.</td></tr>
</table>

APPENDIX B: Flowchart for Determining Wind Load



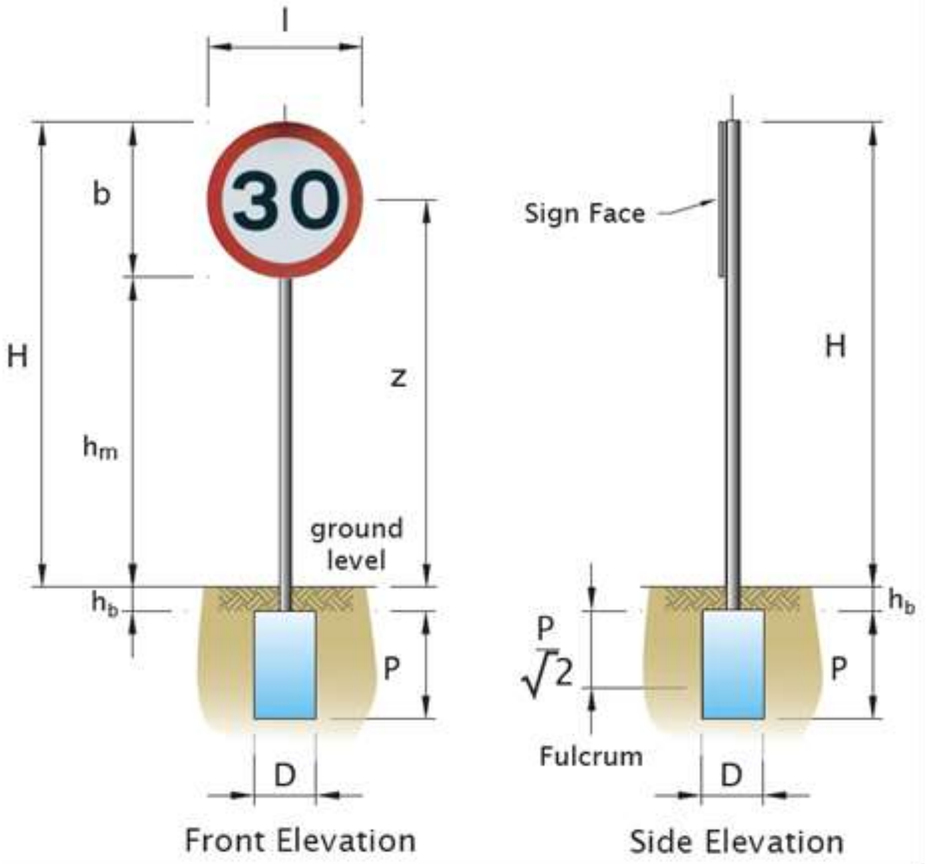
APPENDIX C: EXAMPLES

Introduction

Two examples are examined: a small circular sign on a single support with a planted foundation and a rectangular sign on two supports with a more conventional spread foundation.

For each sign, the wind load is determined using both the simplified method of the National Annex to BS EN 12899-1 (sections 1.1 and 2.1), and the more rigorous and economical alternative method of BS EN 1991-1-4 (sections 1.2 and 2.2). This enables the two approaches to be compared for complexity and results. Whichever method is used to obtain it, the basic wind action is converted into a wind force for each of the states to be examined (sections 1.3 and 2.3). These are used in sections 1.4 and 2.4 for checking the sign supports. In sections 1.5 and 2.5, the chosen foundation is assessed. For the second example the base reinforcement and concrete mix are designed.

EXAMPLE 1: a circular sign with a planted foundation



Location: Surrey, England, 10 km from the coast, not on a very exposed site, cliff or escarpment, nor at a site subject to wind funnelling.

Mounting height above ground	h_m	2.0 m
Width of sign face	l	0.9 m
Height of sign face	b	0.9 m
Total height $= h_m + b$	H	2.9 m
Height to centroid of sign area $= h_m + b/2$	z	2.45 m
Depth of post buried above foundation	h_b	0
Foundation: support planted in plain concrete:		
Minimum diameter of foundation (for CD 354 D is the diameter of the post)	D	
Planting depth of foundation.	P	($\geq 2D$)

Example 1 Section 1: Basic Wind Action using BS EN 12899 National Annex

1.1.1 Look up wind load for site

BS EN 12899-1

H = 2.9 m height, therefore Basic Wind Pressure, w_b = 1.0 kN/m^2 (Used in section 1.3 below.)

Table NA 2

Example 1 Section 2: Basic Wind Action using BS EN 1991-1-4

1.2.1 Look up fundamental value of basic wind velocity

References relate to BS EN 1991-1-4 and its National Annex

$v_{b,0} = v_{b,map} \cdot c_{alt} = 21.5 \times 1.25 = 26.88$ m/s

Eqn NA1

where

$v_{b,map}$ = value of the fundamental basic wind velocity before the altitude correction is applied. (21.5 m/s selected from map)

Figure NA1

c_{alt} = altitude factor

$c_{alt} = 1 + 0.001\,A = 1 + 0.001 \times 250 = 1.25$

Eqn NA2a

A = Altitude of the site (m) above mean sea level (250 m in this example)

1.2.2 Assess Terrain Orography

Not very exposed site on cliff/escarpment or in a site subject to local wind funnelling. Therefore $c_o = 1.0$

Fig NA2

1.2.3 Determine Design Life Requirement

$$c_{prob} = \left(\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K \cdot \ln(-\ln(0.98))}\right)^n = 0.96$$

Eqn 4.2

where

p = design annual probability of exceedence

p = 1/design life = 1/25 = 0.04 (for signs design life is 25 years)

K = Shape parameter = 0.2

n = exponent = 0.5

NA 2.8

1.2.4 Basic Wind Velocity

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}$$

Eqn 4.1

$v_b = 1.0 \times 1.0 \times 26.88 = 26.88$ m/s

where

c_{dir} = directional factor = 1.0

NA 2.6

c_{season} = season factor = 1.0

NA 2.7

10 minute mean wind velocity having probability p for an annual exceedence is determined by: *Clause 4.2 Note 4*

$$v_{b,25\ years} = v_b \cdot c_{prob}$$

$$v_{b,25years} = 26.88 \times 0.96 = 25.80\,\text{m/s}$$

1.2.5 Basic Velocity Pressure

$$q_b = \frac{1}{2}\rho \cdot v_b{}^2 = 0.5 \times 1.226 \times 25.80^2 = 0.408\,\text{kN/m}^2$$ *Eqn 4.10*

where ρ = air density = $1.226\,\text{kg/m}^3$ *NA 2.18*

1.2.6 Peak Velocity Pressure

$$c_{e,flat}(2.45) = 1.66$$ *Fig NA7*

$$q_p(z) = c_{e,flat}(z) \cdot q_b = 1.66 \times 0.408 = 0.68\,\text{kN/m}^2$$ *NA 2.17 eqn NA 3a*

For $z = 2.45\,\text{m}$ where orography is not significant ($c_o = 1.0$) and country terrain category

1.2.7 Basic Wind Pressure

$$w_b = q_p(z) = 0.68\,\text{kN/m}^2$$

This falls within BS EN 12899-1:2007 class WL3, but use of these classes is not recommended.

Example 1 Section 3: Wind Force (for either method)

1.3.1 Determine Force Coefficient c_f

λ = effective slenderness ratio of sign or aspect ratio

$\lambda = l/b$ or $b/l = 0.9/0.9 = 1.0$

BS EN 12899-1

Table NA 2

Therefore $c_f = 1.26$

1.3.2 Calculate Total Wind Force

$F_w = c_s c_d \,.\, c_f \,.\, q_p(z_e) \,.\, A_{ref}$ (A_{ref} = area of sign) *5.3 of EN 1991-1-4*

$c_s c_d$ for signs can be taken as 1.0 *3.15 of this Guide*

$q_p(z_e) = w_b$ (the basic wind pressure determined above)

$F_w = c_f \,.\, w_b \,.\, A_{ref}$

Use $w_b = 1.0\ \text{kN/m}^2$ from section 1.1.1 above for this example.

$$F_w = 1.26 \times 1.0\ \pi\left(\frac{0.9}{2}\right)^2 = 0.80\,\text{kN}$$

1.3.3 Identify Partial Action Factors γ_F

ULS (bending and shear) $\gamma_F = 1.35$ *EN 12899 Table NA 2*

SLS (deflection) $\gamma_F = 1.0$ *Class PAF1 Table 6*

Additional factor γ_{f3}, taken as 1.0, but could alternatively be 1.1 *3.3 of this Guide*

1.3.4 Calculate Design Wind Force on the sign

$F_{w,d} = F_w \,.\, \gamma_F \,.\, \gamma_{f3}$

$F_{w,d}\,(\text{ULS}) = 0.80 \times 1.35 \times 1.0 = 1.08\,\text{kN}$

$F_{w,d}\,(\text{SLS}) = 0.80 \times 1.0 \times 1.0 = 0.80\,\text{kN}$

1.3.5 Wind Force Equivalent to 1-year Return Period

The wind velocity for calculating the temporary deflection (SLS) criterion is 75% of the reference wind velocity, as it is based upon a 1 year mean return period. The 0.96 factor below reverses the c_{prob} conversion from 50 to 25 year return period used above (in 1.2.3). *EN 12899-1 clause 5.4.1 note 1*

$$F_{w,d\,(1\text{ year})} = F_{w,d}\,(\text{SLS}) \times \frac{0.75^2}{0.96^2} = 0.80 \times \frac{0.75^2}{0.96^2} = 0.488\,\text{kN}$$

If the EN 1991-1-4 Basic Wind Pressure from section 1.2.7 above is used, the calculated values are:
$F_{w,d}\,(\text{ULS}) = 0.74\,\text{kN}$ $F_{w,d}\,(\text{SLS}) = 0.55\,\text{kN}$ $F_{w,d\,(1\text{ year})} = 0.34\,\text{kN}$

Example 1 Section 4: Support design (for either method)

1.4.1 Ultimate Design Action

$F_{w,d}\,(\text{ULS}) = 1.08\,\text{kN}$ *1.3.4 above*

1.4.2 Ultimate Action Effects (wind action)

Ultimate design bending moment per post, M_d

M_d = Wind force × lever arm to foundation / number of posts

$= F_{w,d}\,(\text{ULS}) \cdot (z + h_b) / n$ (where n = number of posts)

$= 1.08 \times (2.45 + 0) / 1 = 2.65\,\text{kNm}$

Ultimate design shear per post, V_d, = Wind force / number of posts

$V_d = F_{w,d}\,(\text{ULS}) / n = 1.08 / 1 = 1.08\,\text{kN}$

1.4.3 Ultimate Action Effects (point load)

Investigate the effects of a 0.5 kN point load applied to the post: *BS EN 12899-1 Tables NA 2 & 10 (class PL3)*

Applied Moment:

$M_d = 0.5\ (H + h_b)$

$M_d = 0.5 \times (2.9 + 0)$

$M_d = 1.45\,\text{kNm}$ (note this is less critical than M_d from wind action)

Applied Shear: $V_d = 0.5\,\text{kN}$

$$\text{Torsion} = 0.5\ \frac{L}{2}$$

$$\text{Torsion} = 0.5 \times \frac{0.900}{2} = 0.225\,\text{kNm}$$

1.4.4 Support Properties

Circular Hollow Section, CHS 88.9 × 4.0 (S355 steel) is to be tried.

Steel Building Design, Design data

Characteristic member capacities

$M_{c,Rd} = 10.30\,\text{kNm}$

$V_{c,Rd} = 140.0\,\text{kN}$

1.4.5 Partial Material Factor for Sign Support

$\gamma_m = 1.05$ for steel sections (elongation > 15%)

BS EN 12899-1 Table 7

1.4.6 Ultimate Capacity Check

Ultimate bending capacity = $M_{c,Rd} / \gamma_m = 10.30 / 1.05 = 9.80\,\text{kNm}$

$9.80\,\text{kNm} > 2.65\,\text{kNm}$ *OK*

Ultimate shear capacity = $V_{c,Rd} / \gamma_m = 140.0 / 1.05 = 133.33\,\text{kN}$

$133.33\,\text{kN} > 1.08\,\text{kN}$ *OK*

1.4.7 Combined Bending and Torsion

$$\frac{M}{M_u} + \frac{T}{T_u} \leq 1$$

By inspection, the post sections have sufficient spare capacity to resist combined bending and torsion, since only 50% of available capacity of the post is utilised.

1.4.8 Temporary Deflection Calculation

$F_{w,d\ (1\ \text{year})} = 0.488\ \text{kN}$

Section 1.3.5 above

Uniformly distributed load along sign face = $F_{w,d\ (1\ \text{year})}\ /\ b$

$F_{w,d\ (1\ \text{year})} / b = 0.488 / 0.9 = 0.54\ \text{kN/m}$

where b = height of the sign face

Maximum deflection at top of sign (bending), δ

EN 12899 clause 5.4 & table NA 2

$$\delta = \frac{F_{\mathrm{w,d\,(1\,year)}}/b}{24EI \cdot n}\left[3(H+h_{\mathrm{b}})^4 - 4(h_{\mathrm{m}}+h_{\mathrm{b}})^3(H+h_{\mathrm{b}}) + (h_{\mathrm{m}}+h_{\mathrm{b}})^4\right]$$

Steel Designers' Manual (deflection table for cantilevers)

where

E = Modulus of elasticity of structural steel, taken as 210 x 10^3 N/mm^2

I = Moment of inertia

= 96.3 cm^4 (for CHS 88.9 × 4.0) = 96.3 × 10^4 mm^4

n = number of posts

H, h_b and h_m as illustrated on diagram

Steel Building Design, Design data

$$\Rightarrow \delta = \frac{0.54}{24 \times 210 \times 10^3 \times 96.3 \times 10^4 \times 1} \times$$
$$\left[3 \times (2900+0)^4 - 4 \times (2000+0)^3 \times (2900+0) + (2000+0)^4\right]$$

δ = 15.06 mm

Deflection per linear metre, $\delta' = \dfrac{\delta}{(H+h_b)} = \dfrac{15.06}{(2.9+0)}$ = 5.19 mm/m

Maximum temporary deflection taken as TDB4 = 25 mm/m

BS EN 12899 Tables NA 2 & 11

25 mm/m > 5.19 mm/m *OK*

(If the EN 1991-1-4 method is used, the calculated deflection is 3.74 mm/m)

1.4.9 Conclusion

CHS 88.9 × 4.0 (S355) is sufficient for the design.

A smaller section might also be suitable.

Notes

Design forces may also include the moment and shear force from the wind action on the post(s).

The SLS deflection limits may be more onerous for signs with moving parts.

Note

This calculation is shown done by two differing methods. This first part, 1.5.1 to 1.5.5, is carried out in accordance with PD 6547 for organisations that do not design or manage their structures strictly in accordance with the Design Manual for Roads and Bridges (DMRB), and allows for the inclusion of the concrete or appropriate fill material in the calculation of the ground resistance moment. The second method in 1.5.6 to 1.5.10 follows the current version of DMRB standard CD 354 (revision 1 of March 2020), which does not provide for the inclusion of this surrounding material in the calculation, relying solely on the interaction of the post alone with the surrounding ground.

PART ONE – DESIGN TO PD 6547

1.5.1 Un-factored Design Action

$F_{w,d}\,(\text{SLS}) = 0.80\,\text{kN}$ *Section 1.3.4 above*

1.5.2 Ground Resistance Moment, M_g

Minimum planting depth, $P_{min} = 0.8\,\text{m}$ *EN 40-2 Table 7*

$$M_g = \frac{G \cdot D \cdot P^3}{10}$$

PD 6547 clause 6.3.3

where

G is a factor dependent on the ground in which the support is planted (in kN/m^2 per m). Refer to PD 6547 Table 2 for typical values of G. *PD 6547 clause 6.3.4*

D is the effective diameter of the foundation in the ground (in m), which may be increased to the minimum diameter of the hole (i.e. at the bottom) where an appropriate concrete or backfill is used.

P is the planting depth (in m).

Try foundation design: $D = 0.4\text{m}$, $P = 0.8\,\text{m}$

$P \geq 2D$ *OK* (to ensure the foundation will behave as a planted one.)

Assume 'Poor' ground conditions, $\therefore G$ taken as $230\,\text{kN/m}^2$ per m *PD 6547 Table 2*

$$\Rightarrow M_g = \frac{230 \times 0.400 \times 0.800^3}{10} = 4.71\,\text{kNm}$$

1.5.3 Destabilising Moment

The destabilising moment is calculated about a fulcrum point located at $1/\sqrt{2}$ of the planting depth below ground. *PD 6547 Clause 6.3.1*

$$\therefore \text{ lever arm} = [z + h_b + (\frac{1}{\sqrt{2}}P)]$$

z, h_b and P are as defined previously

M_{DS} = $F_{w,d} \times$ lever arm to fulcrum point / number of posts

$$= F_{w,d} \cdot [z + h_b + (\frac{1}{\sqrt{2}} P)] \ / \ n$$

$$= 0.80 \times (2.45 + 0 + \frac{1}{\sqrt{2}} \times 0.8) / 1 = 2.41 \text{ kNm}$$

$\gamma_{s;d} \cdot M_{DS} = 1.25 \times 2.41 = 3.01\,\text{kNm}$

where $\gamma_{s;d}$ is the factor of safety, 1.25

PD 6547 Clause 6.3.2

1.5.4 Capacity Check

$M_g = 4.71\,\text{kNm} > \gamma_{s;d} \cdot M_{DS} = 3.01\,\text{kNm}$ *OK*

PD 6547 Clause 6.3.4

1.5.5 Conclusion

A foundation of un-reinforced concrete, 0.4 m diameter with 0.8m planting depth is satisfactory. In taking this approach the designer must take note of the following points.

a. All backfilling material is to be placed in 150 mm thick layers and be well compacted;
b. During compaction, care is to be taken to ensure any corrosion protection system on the post is not damaged;
c. Where the hole is back-filled with concrete, the concrete is to extend from the base of the sign post and its diameter must not reduce below that of the design diameter at any point.

PART 2 – DESIGN TO CD 354

1.5.6 The unfactored design action will remain unchanged:

$F_{w,d}$ (SLS) = 0.80 kN

Section 1.3.4 above

CD 354 revision 1 does not allow the effective diameter of the foundation to be increased by the backfill, so the ground resistance moment can only be increased by either increasing the planting depth or using a larger diameter post.

1.5.7 Ground Resistance Moment

Minimum planting depth, $P_{min} = 0.8\,\text{m}$

EN 40-2 Table 7

$$M_g = \frac{G \cdot D \cdot P^3}{10}$$

CD 354 Equation 12.12

where
G is a factor dependent on the ground in which the support is planted (in kN/m^2 per m). Refer to CD 354 Table 12.12 for typical values of G.
D is the diameter of the support (in m).
P is the planting depth (in m).

Try foundation design: $D = 0.089\,\text{m}$, $P = 1.20\,\text{m}$

$P \geq 2D$ *OK* (to ensure the foundation will behave as a planted one.)

Assume 'Poor' ground conditions, $\therefore G$ taken as $230\,\text{kN/m}^2$ per m — *CD 354 Table 12.12*

$$\Rightarrow M_g = \frac{230 \times 0.089 \times 1.200^3}{10} = 3.53\,\text{kNm}$$

1.5.8 Destabilising Moment

The destabilising moment is calculated about a fulcrum point located at $1/\sqrt{2}$ of the planting depth below ground. — *CD 354 Clause 12.10*

$$\therefore \text{ lever arm} = [z + h_b + (\frac{1}{\sqrt{2}}P)]$$

z, h_b and P are as defined previously

$$M_{DS} = F_{w,d} \times \text{lever arm to fulcrum point / number of posts}$$

$$= F_{w,d} \cdot [z + h_b + (\frac{1}{\sqrt{2}}P)] \;/\; n$$

$$= 0.80 \times (2.45 + 0 + \frac{1}{\sqrt{2}} \times 1.2)/1 = 2.64 \text{ kNm}$$

$\gamma_{s;d} \cdot M_{DS} = 1.25 \times 2.64 = 3.30\,\text{kNm}$ — *CD 354 Clause 12.11*

where $\gamma_{s;d}$ is the model factor, 1.25

1.5.9 Capacity Check

$M_g = 3.53\,\text{kNm} > \gamma_{s;d} \cdot M_{DS} = 3.30\,\text{kNm}$ *OK* — *CD 354 Clause 12.15*

1.5.10 Conclusion

A planted foundation 1.2 m deep is satisfactory. The requirements of CD 354 increase the depth of planting by 0.4 m over the design to PD 6547, but they do not depend upon any concrete or backfill surround to the post.

Note

This design method does not apply to foundations on slopes, where the stability of the ground needs to be taken into account. In such instances, specialist geotechnical advice should be sought. An indicative calculation for the same sign placed on a 15° slope is shown in sections 1.6 and 1.7 below — *CD 354 clauses 12.4 and 12.5*

Example 1 Section 6:
Planted Foundation Design to PD 6547 and CD354 on a slope using Method 1

PART ONE – DESIGN TO PD 6547

1.6.1 Un-factored Design Action

$F_{w,d}$ (SLS) = 0.80 kN

Section 1.3.4 above

1.6.2 Ground Resistance Moment, M_g

$$M_g = \frac{G \cdot D \cdot P_{eff}{}^3}{10}$$

PD 6547 clause 6.3.3

Where P_{eff} = Effective planting depth

Try foundation design: $D = 0.4\text{m}$, $P = 1.6\text{m}$

$h_b = 3 \times \tan 15° = 0.80\text{m}$

$P_{eff} = P - h_b = 1.6 - 0.8 = 0.8\text{m}$

Assume 'Poor' ground conditions, $\therefore G$ taken as 230kN/m^2 per m

PD 6547 Table 2

$$\Rightarrow M_g = \frac{230 \times 0.4 \times 0.8^3}{10} = 4.71\,\text{kNm}$$

1.6.3 Destabilising Moment

$\therefore$ lever arm $= [z + h_b + \left(\frac{1}{\sqrt{2}} P_{eff}\right)] / n$

z, h_b and P_{eff} are as defined previously

M_{DS} = $F_{w,d} \times$ lever arm to fulcrum point / number of posts

$= F_{w,d} \cdot [z + h_b + \left(\frac{1}{\sqrt{2}} P_{eff}\right)] / n$

$= 0.80 \times [2.45 + 0.8 + \left(\frac{1}{\sqrt{2}} 0.8\right)] / 1 = 3.2\,\text{kNm}$

$\gamma_{s;d} \cdot M_{DS} = 1.25 \times 3.2 = 4.00\,\text{kNm}$

where $\gamma_{s;d}$ is the safety factor, 1.25

PD 6547 Clause 6.3.2

1.6.4 Capacity Check

$M_g = 4.71\,\text{kNm} > \gamma_{s;d} \cdot M_{DS} = 4.00\,\text{kNm}$ *OK*

PD 6547 Clause 6.3.4

1.6.5 Conclusion

A foundation of un-reinforced concrete, 0.4 m diameter with 1.6m planting depth is satisfactory for the 15° slope.

PART TWO – DESIGN TO CD 354

1.6.6 Un-factored Design Action

The unfactored design action is unchanged: $F_{w,d}\,(SLS) = 0.80$ kN — *Section 1.3.4 above*

1.6.7 Ground Resistance Moment, M_g

$$M_g = \frac{G \cdot D \cdot P_{eff}^{\ 3}}{10}$$

CD 354 Equation 12.12

Where P_{eff} = Effective planting depth

Try foundation design: $D = 0.089\,\text{m}$, $P = 2.1\,\text{m}$

$h_b = 3 \times \tan 15° = 0.80\,\text{m}$

$P_{eff} = P - h_b = 2.1 - 0.80 = 1.3\,\text{m}$

Assume 'Poor' ground conditions, $\therefore G$ taken as $230\,\text{kN/m}^2$ per m — *CD 354 Table 12.12*

$$\Rightarrow M_g = \frac{230 \times 0.089 \times 1.3^3}{10} = 4.50\,\text{kNm}$$

1.6.8 Destabilising Moment

$\therefore$ lever arm $= [z + h_b + \left(\frac{1}{\sqrt{2}}\, P_{eff}\right) / n$

z, h_b and P_{eff} are as defined previously

$M_{DS} = F_{w,d} \times$ lever arm to fulcrum point / number of posts

$= F_{w,d} \cdot [z + h_b + \left(\frac{1}{\sqrt{2}}\, P_{eff}\right)] / n$

$= 0.80 \times [2.45 + 0.8 + \left(\frac{1}{\sqrt{2}}\, 1.3\right)] / 1 = 3.34\,\text{kNm}$

$\gamma_{s;d} \cdot M_{DS} = 1.25 \times 3.34 = 4.18\,\text{kNm}$

where $\gamma_{s;d}$ is the model factor, 1.25 — *CD 354 Clause 12.11*

1.6.9 Capacity Check

$M_g = 4.50\,\text{kNm} > \gamma_{s;d} \cdot M_{DS} = 4.18\,\text{kNm}$ *OK* — *CD 354 Clause 12.15*

1.6.10 Conclusion

A planted foundation 2.1m deep is satisfactory for the 15° slope. The requirements of CD 354 increase the depth of planting by 0.5 m over the design to PD 6547, but they do not depend upon any concrete or backfill surround to the post.

Example 1 Section 7:
Planted Foundation Design to PD 6547 and CD 354 on a slope using Method 2

PART ONE – DESIGN TO PD 6547

1.7.1 Un-factored Design Action

$F_{w,d}$ (SLS) = 0.80 kN *Section 1.3.4 above*

1.7.2 Ground Resistance Moment, M_g

$$M_g = \frac{G \cdot D \cdot P_{eff}^{\;3}}{10}$$

PD 6547 clause 6.3.3

Where P_{eff} = Effective planting depth

Try foundation design: $D = 0.4\,\text{m}$, $P = 1.25\,\text{m}$

$h_b = F_{Slope} \cdot P$

for a slope angle of 15°, $F_{slope} = 0.34$

$h_b = 0.34 \times 1.25 = 0.43$

$P_{eff} = P - h_b = 1.25 - 0.43 = 0.82\,\text{m}$

Assume 'Poor' ground conditions, $\therefore G$ taken as 230 kN/m² per m *PD 6547 Table 2*

$$\Rightarrow M_g = \frac{230 \times 0.4 \times 0.82^3}{10} = 5.07\,\text{kNm}$$

1.7.3 Destabilising Moment

$\therefore$ lever arm $= [z + h_b + \left(\frac{1}{\sqrt{2}} P_{eff}\right)] / n$

z, h_b and P_{eff} are as defined previously

$$M_{DS} = F_{w,d} \times \text{lever arm to fulcrum point / number of posts}$$

$$= F_{w,d} \cdot [z + h_b + \left(\frac{1}{\sqrt{2}} P_{eff}\right)] / n$$

$$= 0.80 \times [2.45 + 0.43 + \frac{1}{2}\ 0.82\ ] / 1 = 2.77\,\text{kNm}$$

$\gamma_{s;d} \cdot M_{DS} = 1.25 \times 2.77 = 3.46\,\text{kNm}$

where $\gamma_{s;d}$ is the model factor, 1.25

PD 6547 Clause 6.3.2

1.7.4 Capacity Check

$M_g = 5.07\,\text{kNm} > \gamma_{s;d} \cdot M_{DS} = 3.46\,\text{kNm}$ *OK*

PD 6547 Clause 6.3.4

1.7.5 Conclusion

A foundation of un-reinforced concrete, 0.4 m diameter with 1.25m planting depth is satisfactory for the 15° slope.

PART TWO – DESIGN TO CD 354

1.7.6 Un-factored Design Action

The unfactored design action is unchanged: $F_{w,d}\,(SLS) = 0.80$ kN

Section 1.3.4 above

1.7.7 Ground Resistance Moment, M_g

$$M_g = \frac{G \cdot D \cdot P_{eff}^{\ 3}}{10}$$

CD 354 Equation 12.12

Where P_{eff} = Effective planting depth

Try foundation design: $D = 0.089$m, $P = 1.95$ m

$h_b = F_{Slope} \cdot P$

for a slope angle of 15°, $F_{slope} = 0.34$

$h_b = 0.34 \times 1.95 = 0.66$

$P_{eff} = P - h_b = 1.95 - 0.66 = 1.29$ m

Assume 'Poor' ground conditions, $\therefore G$ taken as 230 kN/m^2 per m

CD 354 Table 12.12

$$\Rightarrow M_g = \frac{230 \times 0.089 \times 1.29^3}{10} = 4.39\ \text{kNm}$$

1.7.8 Destabilising Moment

$\therefore$ lever arm $= [z + h_b + \left(\frac{1}{\sqrt{2}}\ P_{eff}\right)] / n$

z, h_b and P_{eff} are as defined previously

M_{DS} $= F_{w,d} \times$ lever arm to fulcrum point / number of posts

$= F_{w,d} \times [z + h_b + \left(\frac{1}{\sqrt{2}}\ P_{eff}\right)] / n$

$= 0.80 \times [2.45 + 0.66 + \frac{1}{2}\ 1.29 \quad / 1 = 3.21$kNm

$\gamma_{s;d} \times M_{DS} = 1.25 \times 3.21 = 4.01$ kNm

where $\gamma_{s;d}$ is the model factor, 1.25

CD 354 Clause 12.11

1.7.9 Capacity Check

$M_g = 3.39$ kNm $> \gamma_{s;d} \times M_{DS} = 4.01$ kNm *OK*

CD 354 Clause 12.15

1.7.10 Conclusion

A planted foundation 1.95m deep is satisfactory for the 15° slope. The requirements of CD 354 increase the depth of planting by 0.7 m over the design to PD 6547, but they do not depend upon any concrete or backfill surround to the post.

Notes

The calculations show that planted foundations designed in accordance with CD 354 need to be installed deeper than foundations to PD 6547, which allows the inclusion of acceptable backfill material to increase the effective diameter of the foundation.

In both cases, Method 2 is less conservative than Method 1, resulting in a shallower foundation.

Example 1 Section 8: Points to consider

1.8.1 Passive safety and impact design

This design does not consider impact loading; the scheme designer should assess the need to consider vehicle impact on a case by case basis. This example does not consider passive safety, but where such a need has been identified, the support type required should be provided to the manufacturer.

1.8.2 CE / UKCA Marking and documentation

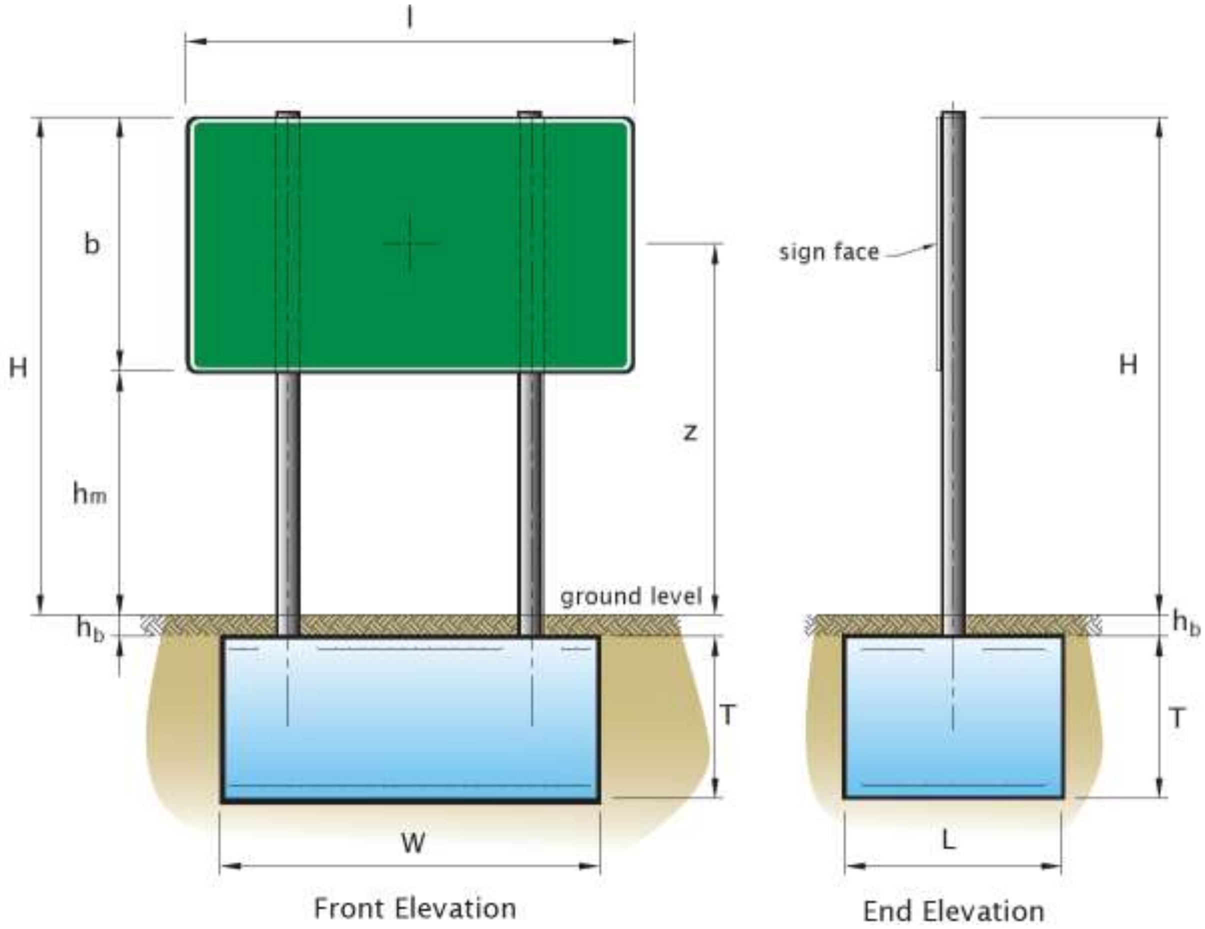
A permanent traffic sign must be CE or UKCA marked in accordance with the Construction Products Regulations. In addition to the complete sign assembly, individual components may be marked:

- the supports;
- the sign plate with stiffening and face material;
- the face material.

The design and the sign plate, reinforcement (channels) and choice of fixings (clips), is usually carried out by the sign manufacturer. To do this, the designer should supply the manufacturer with the design wind loading on the sign face and the number and size of posts that the sign will be supported on.

The manufacturer should supply details including performance classes for each component that has been CE or UKCA marked, in accordance with the requirements set out in annex ZA of BS EN 12899-1. This information, which is often placed on a label affixed to the back of the sign, is essential for the ongoing maintenance of the sign.

EXAMPLE 2: a rectangular sign with a spread foundation



Location Londonderry/Derry, Northern Ireland 7 km from the coast, not on a very exposed site on cliff / escarpment, nor in a site subject to wind funnelling

Mounting height above ground	h_m	1.5 m
Breadth of sign face	l	4.0 m
Height of sign face	b	2.5 m
Total height $= h_m + b$	H	4.0 m
Height to centroid of sign area $= h_m + b/2$	z	2.75 m
Depth of post buried above foundation	h_b	75 mm

Spread foundation:

- width of foundation parallel to sign — W
- length of foundation perpendicular to sign — L
- thickness of foundation — T

Example 2 Section 1: Basic Wind Action using BS EN 12899 National Annex

2.1.1 Look up wind action for site

$H = 4.0$ m, therefore Basic Wind Pressure, $w_b = 1.2\,\text{kN/m}^2$ *EN 12899 Table NA.2*

Example 2 Section 2: Basic Wind Action using BS EN 1991-1-4

2.2.1 Look up fundamental value of basic wind velocity

References relate to BS EN 1991-1-4 and its National Annex

$v_{b,0} = v_{b,map} \,.\, c_{alt} = 26.25 \times 1.217 = 31.95\,\text{m/s}$ *Eqn NA1*

where

$v_{b,map}$ = value of the fundamental basic wind velocity before the altitude correction is applied. (26.25 m/s selected from map) *Figure NA1*

c_{alt} = altitude factor

$c_{alt} = 1 + 0.001\,A = 1 + 0.001 \times 217 = 1.217$ *Eqn NA2a*

A = Altitude of the site above mean sea level (217 m in this example)

2.2.2 Assess Terrain Orography

Not very exposed site on cliff/escarpment or in a site subject to local wind funnelling. Therefore $c_o = 1.0$ *Fig NA2*

2.2.3 Determine Design Life Requirement

$$c_{prob} = \left(\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K \cdot \ln(-\ln(0.98))}\right)^n = 0.96$$ *Eqn 4.2*

where

p = design annual probability of exceedance

p = 1/design life = 1/25 = 0.04 (design life for signs is 25 years)

K = Shape parameter = 0.2 *NA 2.8*

n = exponent = 0.5

2.2.4 Basic Wind Velocity

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0}$$ *Eqn 4.1*

$v_b = 1.0 \times 1.0 \times 31.95 = 31.95\,\text{m/s}$

where

c_{dir} = directional factor = 1.0 *NA 2.6*

c_{season} = season factor = 1.0 *NA 2.7*

10 minute mean wind velocity having probability P for an annual exceedance is determined by: *Clause 4.2 Note 4*

$$v_{b,25\ years} = v_b \cdot c_{prob}$$

$$v_{b,25years} = 31.95 \times 0.96 = 30.67\,\text{m/s}$$

2.2.5 Basic Velocity Pressure

$$q_b = \frac{1}{2}\rho \cdot v_b^2 = 0.5 \times 1.226 \times 30.67^2 = 0.577\,\text{kN/m}^2$$ *Eqn 4.10*

where ρ = air density = $1.226\,\text{kg/m}^3$ *NA 2.18*

2.2.6 Peak Velocity Pressure

$c_e(2.75) = 1.74$ *Fig NA7*

$q_p(z) = c_e(z) \cdot q_b = 1.74 \times 0.577 = 1.00\,\text{kN/m}^2$ *Eqn NA 3a*

For $z = 2.75\,\text{m}$ where orography is not significant ($c_o = 1.0$), country terrain category, flat and $\geq 5\,\text{km}$ from the shore. *NA 2.17*

2.2.7 Basic Wind Pressure

$w_b = q_p(z) = 1.00\,\text{kN/m}^2$

(This is equivalent to BS EN 12899-1:2007 class WL5, but use of these classes is not recommended.)

Example 2 Section 3: Wind Force (for either method)

2.3.1 Determine force coefficient

λ = effective slenderness ratio of sign or aspect ratio
$\lambda = l/b = 4.0 / 2.5 = 1.6$ *BS EN 12899 Table NA 2*

Therefore $c_f = 1.30$

2.3.2 Calculate Total Wind Force

$F_w = c_s c_d \,.\, c_f \,.\, q_p(z_e) \cdot A_{ref}$ (A_{ref} = area of sign) *5.3 of EN 1991-1-4*

$c_s c_d$ for signs can be taken as 1.0 *3.15 of this Guide*

$q_p(z_e) = w_b$

w_b is the basic wind pressure from section 2.1.1 or 2.2.7 above. The section 2.1.1 value of 1.2 kN/m^2 is used for this example.

$F_w = c_f \cdot w_b \cdot A_{ref}$

$F_w = 1.30 \times 1.2 \times 4.0 \times 32.5 = 15.6\,\text{kN}$

2.3.3 Identify Partial Action Factors γ_F

ULS (bending and shear) $\gamma_F = 1.35$

SLS (deflection) $\gamma_F = 1.0$

EN 12899 Table NA 2 & Table 6 (class PAF1)

Additional factor γ_{f3}, taken as 1.0, but could alternatively be 1.1 *3.3 of this Guide*

For stability for spread foundation design refer to commentary above. *5.1 of this Guide*

2.3.4 Calculate Design Wind Force on the sign

$F_{w,d} = F_w \cdot \gamma_F \cdot \gamma_{f3}$.

$F_{w,d}$ (ULS) = 15.6 × 1.35 × 1.0 = 21.1 kN

$F_{w,d}$ (SLS) = 15.6 × 1.0 × 1.0 = 15.6 kN

2.3.5 Wind Force Equivalent to 1-year Return Period

The wind velocity for calculating the temporary deflection (SLS) criterion is 75% of the reference wind velocity, as it is based upon a 1 year mean return period. The 0.96 factor below reverses the c_{prob} conversion from 50 to 25 year return period used above (in 2.2.3).

EN 12899-1 clause 5.4.1 note 1

$$F_{w,d\,(1\,year)} = F_{w,d}\,(SLS) \times \frac{0.75^2}{0.96^2} = 15.6 \times \frac{0.75^2}{0.96^2} = 9.52\,kN$$

If EN 1991-1-4 method is used, the calculated values are:
$F_{w,d}$ (ULS) = 17.6 kN $F_{w,d}$ (SLS) = 13.0 kN $F_{w,d\,(1\,year)}$ = 7.94 kN

Use the above EN 1991 forces in sections 2.4 & 2.5 below.

Example 2 Section 4: Support design (for either method)

2.4.1 Ultimate Design Action

$F_{w,d}$ (ULS) = 17.6 kN

section 2.3.5 above

2.4.2 Ultimate Action Effects

Ultimate design bending moment per post, M_d

M_d = Wind force × lever arm to foundation / number of posts

= $F_{w,d}$ (ULS) · $(z+h_b)$ / n

= 17.6 × (2.75 + 0.075) / 2 = 24.86 kNm

Ultimate design shear per post, V_d = Wind force / number of posts

= $F_{w,d}$ (ULS) / n

= 17.6 / 2 = 8.8 kN

The 0.5 kN point load on the sign is not critical, since it is less than the wind action and there are no torsional effects with 2 posts.

2.4.3 Support Properties

Circular Hollow Section, CHS 168.3 × 5.0 (S355) is to be tried

Characteristic member capacities
$M_{c,Rd}$ = 47.2 kNm
$V_{c,Rd}$ = 335.0 kN

Steel Building Design, Design data

2.4.4 Partial Material Factor for Sign Support

$\gamma_m = 1.05$ for steel sections (elongation > 15%)

BS EN 12899-1 Tables NA2 & 7

2.4.5 Ultimate Capacity Check

Ultimate bending capacity = $M_{c,Rd} / \gamma_m$ = 47.2 / 1.05 = 44.95 kNm

44.95 kNm > 24.86 kNm OK

Ultimate shear capacity = $V_{C,Rd} / \gamma_m$ = 335.0 / 1.05 = 319.05 kN

319.05 kN > 8.80 kN OK

2.4.6 Temporary Deflection Calculations

$F_{w,d\,(1\,year)} = 7.94\,\mathrm{kN}$

section 2.3.5 above

Uniformly distributed load along sign face = $F_{w,d\,(1\,year)} / b$

$F_{w,d\,(1\,year)} / b$ = 7.94 / 2.5 = 3.18 kN/m

where b = height of the sign face

Maximum deflection at top of sign (bending), δ

BS EN 12899 clause 5.4.1 & Table NA.2

$$\delta = \frac{F_{w,d\,(1\,year)}/b}{24EI \cdot n}\left[3(H+h_b)^4 - 4(h_m+h_b)^3(H+h_b) + (h_m+h_b)^4\right]$$

Steel Designers' Manual (Deflection table for cantilevers)

where

E = Modulus of Elasticity of structural steel, taken as $210 \times 10^3\,\mathrm{N/mm^2}$

I = Moment of inertia
= $856\,\mathrm{cm^4}$ (for CHS 168.3 × 5.0) = $856 \times 10^4\,\mathrm{mm^4}$

n = number of posts

H, h_b and h_m as illustrated on diagram

Steel Building Design, Design data

$$\Rightarrow \delta = \frac{3.18}{24 \times 210 \times 10^3 \times 856 \times 10^4 \times 2} \times$$
$$\left[3 \times (4000+75)^4 - 4 \times (1500+75)^3 \times (4000+75) + (1500+75)^4\right]$$

δ = 28.17 mm

Deflection per linear metre, $\delta' = \dfrac{\delta}{(H+h_b)} = \dfrac{28.17}{(4.0+0.075)} = 6.91\,\mathrm{mm/m}$

Maximum temporary deflection taken as class TDB4 = 25 mm/m

25 mm/m > 6.91 mm/m *OK*

BS EN 12899 Tables NA.2 & 11

2.4.7 Conclusion

2 no. CHS 168.3 × 5.0 (S355) supports are sufficient for the design.

A smaller section might also be suitable.

Notes

Design forces may also include the moment and shear force from the wind action on the posts.

Torsion has not been calculated. This should be considered for signs that are fixed eccentrically on the posts, where a significant area could be shielded from the wind or where buffeting from traffic can occur.

The SLS deflection limits may be more onerous for signs with moving parts.

Example 2 Section 5: Foundation Design to BS EN 1992 & BS EN 1997

2.5.1 Characteristic Design Action

$F_{w,d}$ = 13.0 kN

To provide compatibility with BS EN 1997-1, this action is referred to as F_{rep} in the remainder of the calculation.

2.5.2 Assumptions

Assumed Design Bearing Resistances

Assume following BS EN 1997-1 design bearing resistances from Table 5.2 in Section 5 of this Guide, based on available information:

V_d Design Approach 1 Combination 2 (DA1C2): 100 kPa
V_d Design Approach 1 Combination 1 (DA1C1): 135 kPa

Corresponding design soil strengths from table 5.3 in Section 5 of this Guide:

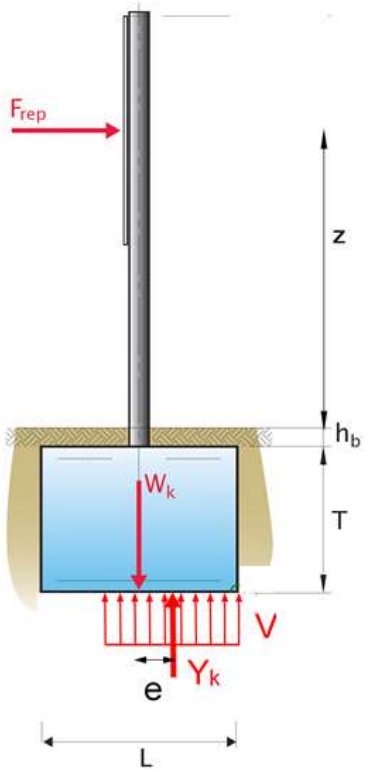
$c_{u;d}$ Design Approach 1 Combination 2 (DA1C2): 18 kPa
$c_{u;d}$ Design Approach 1 Combination 1 (DA1C1): 25 kPa

Presence of fill above foundation

Assume backfill above foundation may be removed during life of structure. Select most onerous characteristic weight with or without backfill for respective load combinations.

2.5.3 Foundation dimensions

w, L and T as illustrated and defined in Figure 5.3 of this Guide.



Characteristic vertical load	W_k
Resultant soil reaction from W_k and F_{rep}	Y_k
Eccentricity of reaction	e

The soil reaction is simplified for this example to a uniform pressure across the portion of the foundation upon which it is taken to act, as explained in paragraph 5.2

Try foundation dimensions: $w = 3.40\,\text{m}$, $L = 2.10\,\text{m}$, $T = 1.50\,\text{m}$
Founding depth is greater than minimum required in this Guide for use of assumed design bearing resistances, see Table 5.3, Section 5. *OK*
Note: most efficient design will minimise w and increase L.

Unit weight of reinforced concrete γ_{con} taken as $24\,\text{kN/m}^3$

Assume unit weight of backfill above foundation of $20\,\text{kN/m}^3$

BS EN 1991-1-1 Table A.1

Assume weight of posts (approx. 1.6 kN) and sign face is not significant compared to foundation.

Characteristic weight of the foundation with backfill above

$W_k = 3.40 \times 2.10 \times (1.50 \times 24 + 0.075 \times 20) = 268\,\text{kN}$

Characteristic weight of the foundation without backfill above

$W_k = 3.40 \times 2.10 \times (1.50 \times 24) = 257\,\text{kN}$

Characteristic Destabilising Moment

$$E_k = F_{rep}\ (z + h_b + T)$$

$$= 13.0 \times (2.75 + 0.075 + 1.50)$$

$$= 56.23\,\text{kNm}$$

2.5.4 Geotechnical Limit States for Consideration

BS EN 1997-1:2004 clause. 6.2 requires that an appropriate list is compiled from the following limit states for the design of spread foundations:

1. loss of overall stability;
2. bearing resistance failure, punching failure, squeezing;
3. failure by sliding;
4. combined failure in the ground and in the structure;
5. structural failure due to foundation movement;
6. excessive settlements;
7. excessive heave due to swelling, frost and other causes;
8. unacceptable vibrations.

Given the low complexity and the low geotechnical risk associated with the design, Geotechnical Category 1 is considered appropriate as defined in BS EN 1997-1 Cl. 2.1(14) to 2.1(16). Where this is not considered appropriate, more rigorous assessment would be necessary. Given the negligible risk of significant ground movements, limit states 5, 7 and 8 are not considered to require further consideration.

As ground movements are expected to be insignificant and any soil-structure interaction is expected to be simple in nature, limit state 4 is addressed through consideration of the GEO and STR limit states. As stated in Section 5 of this Guide, for the range of design bearing resistances presented in Table 5.2 checking settlement, i.e. limit state 6, is not considered to be required for spread foundations supporting only sign loading.

Limit states 2, 3 and 1 are considered in this section.

2.5.5 GEO Limit State: Bearing and sliding

Bearing: $V_d \leq R_{d;v}$ where:

V_d = Sum of the design values of the effects of vertical actions.

$R_{d;v}$ = Bearing capacity of the ground into which the foundation is constructed.

EN 1997 Eqn 6.1

Sliding: $H_d \leq R_d$ where:

R_d = $V'_d \tan\delta_d$ or $A\, c_{u;d}$ and $R_d \leq 0.4V_d$

Note that these requirements will be satisfied if the more onerous limits on horizontal load requirements related to bearing are satisfied, see Section 5 of this Guide.

EN 1997 Eqn 6.2 Eqn 6.3a Eqn 6.4a Eqn 6.4b

Design Approach 1 Design Combination 1: A1 + M1 + R1

where "+" implies: *to be combined with*

EN 1997 2.4.7.3.4.2 Eqn 2.5

DA1C1, permanent load considered as stabilising

(Unlikely to govern bearing capacity but likely to govern eccentricity and hence foundation dimensions.)

Check eccentricity

Destabilising Moment, E_d : Variable Unfavourable

γ_Q = 1.50

BS EN 1990 Table NA.A1.2(B)

E_d = E_k . γ_F

BS EN 1997 2.4.6.1

= 56.23 × 1.50

= 83.34 kNm

Weight of foundation without backfill (assume backfill may be removed during life of structure and omission will maximise eccentricity)

BS EN 1990 Table NA.A1.2(B)

$\gamma_{G,inf}$ = 1.0 (Permanent load stabilising)

BS EN 1997 2.4.6.1

W_d = $W_k \cdot \gamma_G$ = 257 kN

e = E_d/W_d = 83.34 kNm / 257 kN = **0.33 m (maximum eccentricity)**

$L/3$ = 2.1 / 3 = 0.7 m

EN 1997 6.5.4

$e \leq L/3$ *Eccentricity OK*

Check Bearing

Design Bearing Pressure

L' = L – 2e = 1.44 m

$L' \geq 0.5$ m *OK* (see requirement in Table 5.3, Section 5 of Guide)

V_d = $W_d / (L' \cdot w)$ = 257 kN / (1.44 × 3.4) m = 257 kN / 4.91 m^2

V_d = 52.4 kPa

Compare with Assumed Design Bearing Resistance (DA1C1) of 135 kPa

52.4 kPa ≤ 135 kPa

$V_d \leq R_{d;v}$ *Bearing OK* (**39%** utilisation)

BS EN 1990 Table NA.A1.2(B)

Check horizontal load requirements/ Sliding check

BS EN 1997 2.4.6.1

F_{rep} = 13kN

γ_Q = 1.5

H_d = $F_{rep} \cdot \gamma_Q$ = 19.5 kN

$c_{u;d}$ = 25 kPa (see Assumptions above)

EN 1997 6.5.3

Sliding check will be satisfied if horizontal load requirement for assumed design bearing resistance is met, see Section 5 of this Guide:

Cohesive soil:

$H_d / (L' \cdot w \cdot c_{u;d})$ = 19.5 / (1.44 × 3.4 × 25) = **0.16**

$H_d / (L' \cdot w \cdot c_{u;d}) \leq 0.36$ *OK (Therefore sliding also OK)*

Table 5.3, B3

Check of horizontal load requirement H_d / $W_d \leq 0.15$, related to granular soil bearing capacity, not required in DA1C1 (see Section 5 of this Guide.)

DA1C1 permanent load considered as destabilising

Check eccentricity

Destabilising Moment, E_d : Variable Unfavourable

BS EN 1990 Table NA.A1.2(B)

BS EN 1997 2.4.6.1

$\gamma_Q = 1.50$

$E_d = E_k \cdot \gamma_F$

$= 56.23 \times 1.50$

$= 83.34\,\text{kNm}$

Weight of foundation with backfill

BS EN 1990 Table NA.A1.2(B)

BS EN 1997 2.4.6.1

$W_k = 3.40 \times 2.10 \times (1.50 \times 24 + 0.075 \times 20) = 268\,\text{kN}$

$\gamma_{G,sup} = 1.3$ (Permanent load de-stabilising)

$W_d = W_k \cdot \gamma_G = 268\,\text{kN} \times 1.3 = 382\,\text{kN}$

$e = E_d / W_d = 83.34\,\text{kNm} / 382\,\text{kN} = \mathbf{0.24\,m}$

$L/3 = 2.1 / 3 = 0.7\,\text{m}$ — *EN 1997 6.5.4*

$e \leq L/3$ *Eccentricity OK*

$L' = L - 2e = 1.62\,\text{m}$

$L' \geq 0.5\,\text{m}$ *OK* (see requirement in Table 5.3, Section 5 of Guide)

Check bearing

Design Bearing Pressure

$V_d = W_d / (L' \cdot w) = 348\,\text{kN} / (1.62 \times 3.4)\,\text{m} = 268\,\text{kN} / 5.49\,\text{m}^2$

$V_d = 63.4\,\text{kPa}$

Compare with Assumed Design Bearing Resistance (DA1C1) of 135 kPa

63.4 kPa ≤ 135 kPa

$V_d \leq R_{d;v}$ *Bearing OK* (**47%** utilisation)

DA1C1 bearing utilisation is higher in this case considering permanent load as de-stabilising. (Note same conclusion may not hold at higher eccentricity.)

Check horizontal load requirements/ Sliding check

BS EN 1990 Table NA.A1.2(B)

BS EN 1997 2.4.6.1

$F_{rep} = 13\,\text{kN}$

$\gamma_Q = 1.5$

$H_d = F_{rep} \cdot \gamma_Q = 19.5\,\text{kN}$

$c_{u;d} = 25\,\text{kPa}$ (see Assumptions above)

Sliding check will be satisfied if horizontal load requirement for assumed design bearing resistance is met, see Section 5 of this Guide: — *EN 1997 6.5.3*

Cohesive soil:

$H_d / (L' \cdot w \cdot c_{u;d}) = 19.5 / (1.62 \times 3.4 \times 25) = \mathbf{0.14}$

$H_d / (L' \cdot w \cdot c_{u;d}) \leq 0.36$ *OK (Therefore sliding also OK)* *Table 5.3, B3*

(Check of horizontal load requirement $H_d / W_d \leq 0.15$ related to granular soil bearing capacity not required in DA1C1, see Section 5 of this Guide.)

2.5.6 Design Approach 1 Design Combination 2: A2 + M2 + R1

EN 1997 2.4.7.3.4.

Destabilising Moment, E_d : Variable Unfavourable

$E_k = F_{rep} \ (z + h_b + T)$

$= 13.0 \times (2.75 + 0.075 + 1.50)$

$= 56.23\,\text{kNm}$

$\gamma_Q = 1.30$

BS EN 1990 Table NA.A1.2(C)

BS EN 1997 2.4.6.1

$E_d = E_k \, . \, \gamma_F$

$= 56.23 \times 1.30 \qquad = 73.09\,\text{kNm}$

Weight of foundation with backfill

$W_k = 3.40 \times 2.10 \times (1.50 \times 24 + 0.075 \times 20) = 268\,\text{kN}$

$\gamma_G = 1.0$

BS EN 1990 Table NA.A1.2(C)

$W_d = 268\,\text{kN}$

BS EN 1997 2.4.6.1

$e = E_d / W_d = 73.09\,\text{kNm} / 268\,\text{kN} = \mathbf{0.27\,m}$

$L/3 = 2.1 / 3 = 0.7\,\text{m}$

$e \leq L/3$ *Eccentricity OK*

EN 1997 6.5.4

$L' = L - 2e = 1.55\,\text{m}$

$L' \geq 0.5\text{m}$ *OK* (see requirement in Table 5.3, Section 5 of Guide)

Check bearing

Design Bearing Pressure

$V_d = W_d / (L' \cdot w) = 268\,\text{kN} / (1.55 \times 3.4)\,\text{m} = 268\,\text{kN} / 5.28\,\text{m}^2$

$V_d = 50.7\,\text{kPa}$

Compare with assumed design bearing resistance (DA1C2) of 100 kPa

$50.7\,\text{kPa} \leq 100\,\text{kPa}$

$V_d \leq R_{d;v}$ *OK* (**51%** utilisation) **(maximum bearing utilisation)**

Bearing is governed by DA1C2 rather than DA1C1.
(This is often the case unless eccentricity is high.)

Check horizontal load requirements/ Sliding check

BS EN 1990 Table NA.A1.2(C)

$F_{rep} = 13\,\text{kN}$

BS EN 1997 2.4.6.1

$\gamma_Q = 1.3$

H_d $= F_{rep} \cdot \gamma_Q = 16.9\,kN$

$c_{u;d}$ $= 18\,kPa$ (see Assumptions above) *EN 1997 6.5.3*

Sliding check will be satisfied if horizontal load requirements for assumed design bearing resistance are met, see Section 5 of this Guide:

Cohesive soil:

$H_d / (L' \cdot w \cdot c_{u;d}) =$ 16.9 / (1.55 × 3.4 × 18) = **0.18**
(maximum cohesive soil horizontal load utilisation)

$H_d / (L' \cdot w \cdot c_{u;d}) \leq 0.36$ *OK (Therefore sliding also OK)* *Table 5.3, B3*

(DA1C2 governs horizontal load check, as expected, due to inclusion of partial factors on both action and ground strength.)

Granular soil:

H_d / W_d = 16.9 / 268 = **0.06**

$H_d / W_d \leq 0.15$ *OK (Therefore sliding also OK)*

2.5.7 EQU Limit state

EQU variable load factor γ_Q of 1.5 is identical to DA1C1 but stabilising permanent load factor $\gamma_{G,inf}$ is 0.9 compared to 1.0 in DA1C1 (with permanent load considered stabilising). *EN 1997 Eqn 2.4*

BS EN 1990 Table NA.A1.2(A)

Eccentricity in EQU limit state is therefore eccentricity in DA1C1 (permanent stabilising) calculation divided by 0.9:

e = 0.33 / 0.9 = 0.36 m

$L/2$ = 2.1 / 2 = 1.05 m

$e \leq L/2$ EQU *OK*

(As e is required to be limited to a maximum of 0.33L in DA1C1, the maximum e in EQU will be 0.33L/0.9 = 0.37L, i.e. less than 0.5L and therefore EQU is acceptable by inspection. EQU may be relevant where a bearing check is not required, e.g. foundation on strong rock.)

2.5.8 Conclusion

A foundation with W = 3.40 m, L= 2.10 m and T = 1.50 m is sufficient for the design.

A smaller foundation, particularly one with a reduced thickness, T, might also be suitable.

Refer to Example 1 for planted bases, which are often an economical alternative.

Example 2 Section 6: Reinforced and Plain Concrete Foundation Design to BS EN 1992

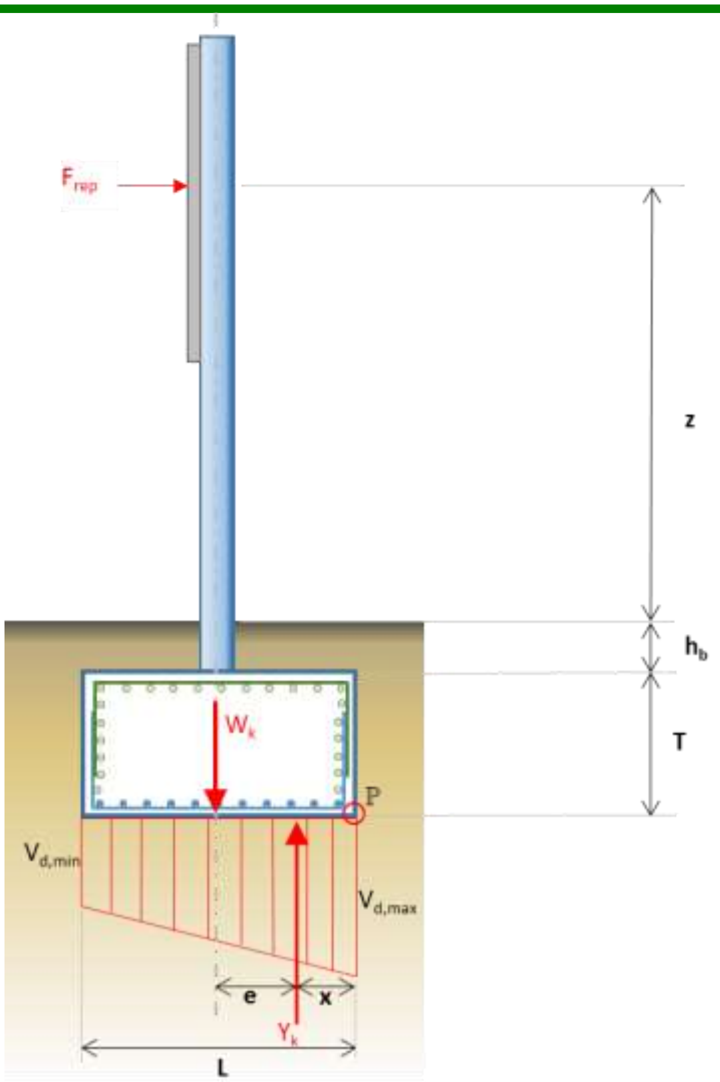
The following example extends the calculations for Example 2 to include the design of the foundation reinforcement and concrete specification. This is illustrative and provided for information only. The advice of a competent structural engineer should be sought during the design process to ensure site-specific requirements are met.

The sections are:

2.5.9 Basic Input Information

DMRB CD 354 clause 12.2 requires foundations to be of reinforced concrete and the design based upon the methods given in BS EN 1997-1 and BS EN 1992. This standard applies to trunk roads and other situations where the specification requires it.

The following foundation geometry and loading has been assumed:



γ_{con}	Concrete density	24 kN/m³	*Table A.1 BS EN 1991-1-1:2002*
z	Height of centre of sign above ground	2.75 m	
h_b	Soil fill height above foundation base	0.075 m	
T	Foundation thickness	1.50 m	
L	Foundation length perpendicular to sign	2.10 m	
W	Foundation width parallel to sign	3.40 m	
W_k	Foundation weight = $\gamma_{con} \cdot L \cdot T \cdot W$	257.04 kN	
F_{rep}	Characteristic wind force from sign (see 2.5.1)	13.00 kN	
	Partial Safety Factor Class	PAF1	*Table NA.2 BS EN 12899-1:2007*
$\gamma_{G,j,sup}$	Load factor for unfavourable permanent loads	1.20	*Table 6 BS EN 12899-1:2007*
$\gamma_{Q,1}$	Load factor for unfavourable variable loads	1.35	*Table 6 BS EN 12899-1:2007*
γ	Load factor for favourable loads	1.00	*Table A1.2(B) BS EN 1990:2002*

2.5.10 Summary of Effects

Following BS EN 1997-1:2004+A1:2013 cl 2.4.7.1, the ultimate limit state design of the foundation reinforcement shall be verified against the following:

> *STR Limit state. Internal failure or excessive deformation of the structure or structural members, including footings, piles, basement walls, etc., where the strength of construction materials of the structure governs.*

Resistance shall be verified in accordance with BS EN 1997-1 cl 2.4.7.3:

$E_d \leq R_d$

where:

E_d is the design value of the effect of actions;

R_d is the design value of the corresponding resistance

For the design of the reinforcement, the overturning moment is assumed to cause the worst effect. Base sliding should also be checked (see 2.5.5) but this will usually be found to be less onerous than the overturning check. Bearing capacity should also be checked (see 2.5.5).

Note that the bearing pressure distribution and magnitude calculated in this section will differ from that in the preceding geotechnical calculations as the partial factors used are different for geotechnical checks and reinforcement design. In particular, the partial factor for unfavourable permanent loads, is taken as 1.20 rather than 1.35.

To determine the design moment in the foundation, Design Approach 1 Combination 1 has been adopted as follows (BS EN 1997-1 cl. 2.4.7.3.4.2).

The destabilizing moment, E_d:

$E_d = \gamma_{Q,1} \cdot F_{rep}.(z + h_b + T)$

$= 1.35 \times 13.0 \times (2.75+0.075+1.50)$

$= 75.90$ kNm

The restoring moment (about ℙ), R_d, ignoring the benefit of the soil weight above the foundation and the self-weight of the sign and post:

$R_d = \gamma \cdot W_k \cdot L/2$

$= 1.00 \times 257.04 \times (2.10 / 2)$

$= 269.89$ kNm

The position of the resultant reaction acting on the base, x:

$x \quad = (R_d – E_d) / W_k$

$= (269.89 – 75.90) / 257.04$

$= 0.755\,m$

The eccentricity from the centre of the base for the resultant reaction, e:

$e \quad = L/2 – x$

$= 2.10 / 2 – 0.755$

$= 0.295\,m$

$L/6 \quad = 2.10 / 6 = 0.350\,m$

Note that e can also be calculated as E_d / W_d where $W_d = W_k.\,\gamma_{G,inf}$

Therefore, the reaction is within the middle third of the base since $e < L / 6$.

The earth pressures at the edge of the base are determined as follows:

Maximum bearing pressure, $V_{d,max}$

$V_{d,max} \quad = \{W_k.\gamma_{G,j,sup}.(1 + 6\,e / L)\} / WL$

$= \{257.04 \times 1.20 \times [1 + 6 \times (0.295/2.10)]\} / (3.40 \times 2.10)$

$= \{257.04 \times 1.20 \times 1.844\} / 7.14$

$= 79.65\,kPa$

Minimum bearing pressure, $V_{d,min}$

$V_{d,min} \quad = \{W_k.\gamma_{G,j,sup}.(1 – 6\,e / L)\} / WL$

$= \{257.04 \times 1.20 \times [1 – 6 \times (0.295/2.10)]\} / (3.40 \times 2.10)$

$= \{257.04 \times 1.20 \times 0.156\} / 7.14$

$= 6.75\,kPa$

Average bearing pressure, $V_{d,ave}$

$V_{d,ave} \quad = (V_{d,max} + V_{d,min}) / 2$

$= (79.65 + 6.75) / 2$

$= 43.20\,kPa$

Therefore, the moment at the centre of the foundation, $M_{L/2}$, resulting from the earth pressures:

$$M_{L/2} = \{L^2 \cdot (V_{d,ave} + 2\ V_{d,max})\} / 24$$

$$= \{2.10^2 \times (43.20 + 2 \times 79.65)\} / 24$$

$$= \{4.41 \times 202.50\} /24$$

$$= 37.21\,\text{kNm/m}$$

Considering bearing pressure alone, design the foundation for an ultimate moment of 37.21 kNm/m

This more conservative design moment is used in the illustrative reinforcement example calculation below.

However, the self-weight of the foundation acts to counter this moment, so it is possible and safe to reduce the design moment by ignoring the component of the soil pressure generated by the self-weight of the base, $V_{d,base}$

$$V_{d,base} = W_k / L \cdot W$$

$$= 257.04 / (3.40 \times 2.10)$$

$$= 36.00\,\text{kPa}$$

Therefore, the reduced moment at the centre of the foundation, $M_{L/2}$, resulting from the reduced earth pressure becomes:

$$M_{L/2} = \{L^2 \cdot (V_{d,ave} + 2\ V_{d,max} - 3\ V_{d,base})\} / 24$$

$$= \{2.10^2 \times (43.20 + 2 \times 79.65 - 3 \times 36.00)\} / 24$$

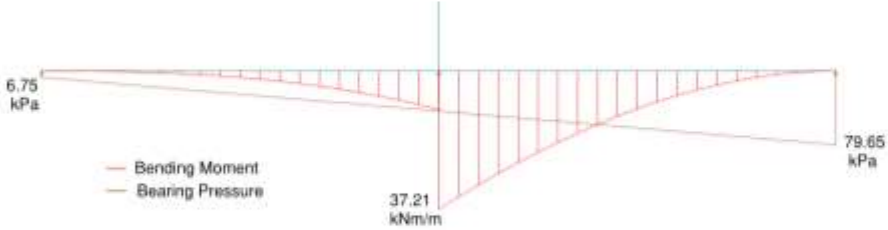
$$= \{4.41 \times 94.5\} / 24$$

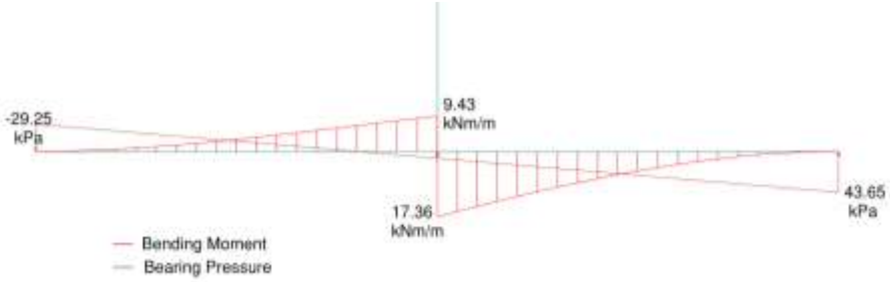
$$= 17.36\,\text{kNm/m}$$

Considering bearing pressure adjusted for self-weight, design the foundation for an ultimate moment of 17.36 kNm/m

This more economical design moment is used in the illustrative plain concrete example calculation below.

The bearing pressure and resulting bending moment assumed in each of these scenarios is shown diagrammatically below.





Idealised structure: bearing pressure and bending moment for each scenario
Upper diagram uses the full bearing pressure;
lower diagram adjusts for the effect of the self-weight.

2.5.11 Plain concrete design

Many sign foundations are installed without reinforcement, particularly those that are not more than twice as long (perpendicular to the sign) as their vertical thickness,

i.e. $L \leq 2\,T$

However, such a foundation does not comply with CD 354 unless specific dispensation has been obtained from the client.

Where it is permissible to use plain concrete foundations, design should be to section 12 of BS EN 1992-1-1:2004.

As in the reinforcement design below, Grade C35/45 concrete has been assumed. This has a typical mean tensile strength of 3.21 N/mm^2, and a 5% fractile tensile strength of 70% of this:

$$f_{ctk,0.05} = 0.7 \times 3.21 = 2.25\ \text{N/mm}^2$$

The design tensile stress limit for plain concrete, $f_{ctd,pl}$:

BS EN 1992-1-1
Eqns. 3.16 &12.1

$$f_{ctd,pl} = \alpha_{ct,pl} \cdot f_{ctk,0.05} / \gamma_c$$

where $\alpha_{ct,pl} = 0.8$ and $\gamma_c = 1.50$ *Tables NA.1 & 2.1N*

$$f_{ctd,pl} = 0.8 \times 2.25 / 1.50$$

$$f_{ctd,pl} = 1.20\,\text{N/mm}^2 = 1.20\,\text{MPa}$$

BS EN 1992 allows us to consider the concrete as uncracked for the ULS, and therefore able to resist limited tension, if the principal tensile stress does not exceed $f_{ctd,pl}$ *12.6.3(3)*

This is more conservative, so safer than using the *Tensile Strength of Cracking* of Grade C35/45 concrete of 1.34 N/mm^2.

Using elastic design, as concrete is unable to redistribute stress, the maximum stress in a block of material T high, by unit width is:

$$\sigma_{t,max} = M \cdot y / I$$

where:

M is the maximum moment in the slab, per unit width = $M_{L/2}$,

$M_{L/2}$ = 17.36 kNm/m
(adjusted for the effect of the self-weight of the slab), as above

y is the distance from the neutral axis to the extreme fibre:

$$y = T / 2$$

$$y = 1.5 / 2 \quad = \quad 0.75\,\text{m}$$

I is the second moment of area:

$$I = 1 \times T^3 / 12 \quad \text{(for unit width)}$$

$$= 1 \times 1.50^3 / 12 \quad = \quad 0.281\,\text{m}^4/\text{m}$$

$$\sigma_{t,max} = M \cdot y / I$$

$$= 17.36 \times 0.75 / 0.281$$

$$= 46.3\,\text{kN/m}^2 \quad = 0.0463\,\text{MPa}$$

Compare $\sigma_{t,max}$ with f_{ctd} :

$$0.0463\,\text{MPa} < 1.20\,\text{MPa}$$

The extreme fibre stress is well within the tensile capacity of the concrete, so this foundation would be satisfactory without reinforcement (if the specification permits).

For cases where the foundation length, L, is more than twice the vertical thickness, T, it is also necessary to check for shear.

2.5.12 Reinforcement design

For the subsequent reinforcement calculations, the more conservative design moment based upon the full bearing pressure is used. But using the lower value that takes into account the effect of the self-weight would also be valid.

For this example calculation therefore, design the foundation for an ultimate moment of 37.21 kNm/m

Grade C35/45 concrete has been assumed.

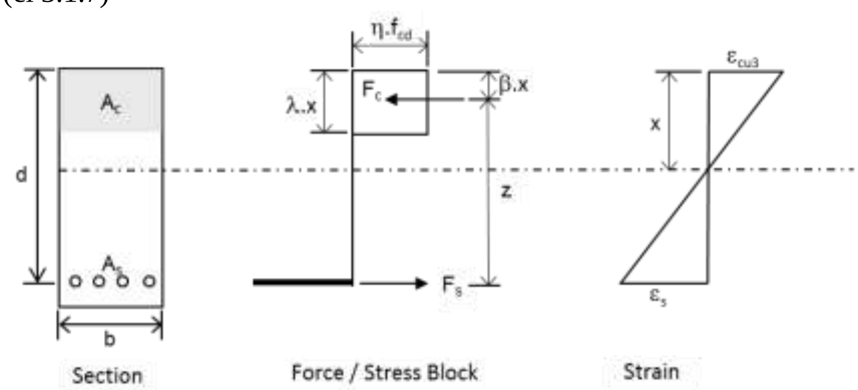
Reinforcement design below is based on 1m length of foundation i.e. b = 1000 mm

References are to BS EN 1992-1-1:2004+A1:2014 and the corresponding National Annex unless stated otherwise.

The following input information has been adopted:

c_{nom}	concrete cover to reinforcement A_s	50 mm	*BS EN 1992-1-1 4.4.1*
φ_s	Assumed reinforcement diameter	16 mm	
T	Foundation thickness	1,500 mm	
λ	Height of compression zone factor	0.8	*BS EN 1992-1-1 Eqn. 3.19*
β	Centroid of compression zone factor = λ/2	0.4	
ε_{cu3}	Ultimate compressive strain	0.0035	*BS EN 1992-1-1 Table 3.1*
η	effective strength coefficient	1.0	*BS EN 1992-1-1 Eqn. 3.21*
f_{ck}	Concrete cylinder strength	35 N/mm^2	*C35/45 Grade Concrete*
α_{cc}	Strength coefficient for long term effects	0.85	*N.A. to BS EN 1992-1-1*
γ_c	Partial safety factor for concrete	1.50	*BS EN 1992-1-1 Table 2.1N*
f_{yk}	Reinforcement strength	500 N/mm^2	
E_S	Young's Modulus for steel	200 kN/mm^2	*BS EN 1992-1-1 3.2.7*
γ_s	Partial safety factor for steel	1.15	*BS EN 1992-1-1 Table 2.1N*

Using the rectangular stress block method in EN 1992-1-:2004+A1:2014 (cl 3.1.7)



The effective depth, *d*, to the reinforcement:

$$d = T - c_{nom} - \varphi_s / 2$$

$$= 1{,}500 – 50 – (16/2)$$

$$= 1{,}442\,\text{mm}$$

The compressive stress limit of the concrete, f_{cd}:

$$f_{cd} = \alpha_{cc} \cdot f_{ck} \;/\; \gamma_c$$

BS EN 1992-1-1 Eqn. 3.15

$$= 0.85 \times 35 / 1.50$$

$$= 19.83\,\text{N/mm}^2$$

This leads to an average compressive stress, f_{av}, of:

$$f_{av} = \lambda \cdot \eta \cdot f_{cd}$$

$$= 0.8 \times 1.0 \times 19.83$$

$$= 15.87\ \text{N/mm}^2$$

The relative value of the compressive stress, K:

$$K = M_{L/2} / (b \cdot d^2 \cdot f_{av})$$

$$= 37.21 \times 10^6 / (1{,}000 \times 1{,}442^2 \times 15.87)$$

$$= 0.00113$$

The neutral axis depth, x, is limited as follows:

$$x \leq d \cdot \{1 / (f_{yk} / (\gamma_s \cdot E_s \cdot \varepsilon_{cu3}) + 1)\}$$

This can be rearranged to:

$$x/d \leq 1 / (f_{yk} / (\gamma_s \cdot E_s \cdot \varepsilon_{cu3}) + 1)$$

$$\leq 1 /(500 /(1.15 \times 200{,}000 \times 0.0035) + 1)$$

$$\leq 1 /(500 / 805) + 1$$

$$\leq 1 /(0.621 + 1)$$

$$\leq 0.617$$

The neutral axis can be determined using the following formula:

$$x = d \cdot \{1 - (1 - 4 \cdot \beta \cdot K)^{0.5}\} / (2 \cdot \beta)$$

This can be rearranged to:

$$x/d = \{1 - (1 - 4 \cdot \beta \cdot K)^{0.5}\} / (2 \cdot \beta)$$

$$= \{1 - (1 - 4 \times 0.4 \times 0.00113)^{0.5}\} / (2 \times 0.4)$$

$$= \{1 - (0.998)^{0.5}\} / 0.8$$

$= 0.00113$ Therefore within limit, take $x/d = 0.00113$

The lever arm, z, is usually given an upper limit as follows:

$$z \leq 0.95\ d$$

The lever arm can be determined using the following:

$$z/d = 1 - (\beta \cdot x/d)$$

$$= 1 - 0.4 \times 0.00113$$

$$= 1 - 0.000451$$

$$= 0.999 > 0.95 \quad \text{Therefore take } z/d = 0.95$$

Hence determine the lever arm as:

$$z = 0.95d$$

$$= 0.95 \times 1{,}442$$

$$= 1{,}370\,\text{mm}$$

The tensile reinforcement required, A_s

$$A_s = M_{L/2} \cdot \gamma_s / f_{yk} \cdot z$$

$$= (37.21 \times 10^6) \times 1.15 / \{500 \times 1{,}370\}$$

$$= 62.5\,\text{mm}^2 / \text{m}$$

The minimum reinforcement diameter required in accordance with *clause 9.8.2.1* is 8mm.

The minimum reinforcement requirement should also be checked in accordance with crack control criterion.

Assuming 16mm diameter bars at 200mm spacing (provides 1005mm²/m).

2.5.13 *Crack control requirements:*

Whilst it may not be necessary to provide crack control reinforcement in foundations of the size of sign this Guide covers, the process is included here for completeness and to provide a method of design for larger signs.

As well as satisfying minimum reinforcement requirements, for strict compliance with the relevant codes and durability requirements, it is also necessary to check crack widths that may result from restrained deformation.

References are to CIRIA C766 *Control of Cracking caused by restrained deformation in concrete* (2018), unless stated otherwise.

The following input information has been used (also see 2.5.15)

	Concrete grade -cylinder / cube	C35/45	
f_{ck}	Characteristic cylinder strength	$35\,N/mm^2$	
$f_{ck,\,cube}$	Characteristic cube strength	$45\,N/mm^2$	
$E_{cm,\,base}$	Mean value of modulus of elasticity $E_{cm} = 22 \times ((f_{ck} + 8) / 10)^{0.3}$	$34.1\,kN/mm^2$	Table 3.1 BS EN 1992-1-1
E_{cm}	Mean value of modulus of elasticity for flint gravel (adjustment factor = 1.1)	$37.5\,kN/mm^2$	Table 4.14
γ_c	Partial safety factor for concrete	1.5	Table 2.1N BS EN 1992-1-1
f_{yk}	Reinforcement strength	$500\,N/mm^2$	
E_s	Reinforcement Young's Modulus	$200\,kN/mm^2$	3.2.7 - EN1992-1-1
γ_s	Partial safety factor for steel	1.15	Table 2.1N - EN1992-1-1
α_c	Coefficient of thermal expansion	$12 \times 10^{-6}/\,°C$	Section 3.7
φ_s	Steel reinforcement diameter	16 mm	As adopted in the reinforcement design
s	Reinforcement spacing	200 mm	
$A_{s\ prov}$	Reinforcement provision = $\pi.\varphi_s^{2} \cdot b / \{4 \cdot s\}$	$1{,}005\,mm^2/m$	
b	Section width	1,000 mm	Considering a 1 m wide section
h	Section thickness	1,500 mm	
c_{nom}	Nominal cover	50 mm	Assumed above
d	Effective depth	1,442 mm	Determined above
w	Limiting crack width	0.30 mm	Table NA.4 NA to EN1992-1-1

Construction conditions – values assumed to give rise to maximum crack widths

	Type of restraint	Internal	Section 3.2.2
	Formwork type	Plywood	Table 3.3
	Aggregate Type	Flint Gravel	Tables 4.4, 4.14 & 4.15
	Concreting period	Summer	Table 3.3
	Cement Type	Class N	Section 3.1.2 (6) EN1992-1-1

To predict the risk of **early-age** cracking and reinforcement requirement for control, the following information is required:

ΔT_1	Temperature drop	56 °C	Section 3.7, Table 3.3
k	Coefficient for stress distribution	1.0	Section A7.4.4
k_c	Coefficient for non-uniform stress	0.5	Section A7.4.4
k_1	Coefficient for reinforcement bond properties	1.143	Section 3.5.2 (= 0.8/0.7)
K_{c1}	Coefficient for creep stress relaxation	0.65	Table 1 & Section 4.9.2
R_1	Restraint factor for early thermal cracking	0.42	Section 4.8.5
$f_{ctm(3)}$	Concrete early tensile strength (C35/45)	$1.92\,N/mm^2$	Section 4.9 Table 4.13
$f_{ctk,0.05(tc)}$	Tensile strength at cracking (C35/45)	$1.34\,N/mm^2$	Section 4.9 Table 4.13
$f_{ct,r(3)}$	Lower characteristic tensile strength	$1.34\,N/mm^2$	Section 3.4.2
ε_{ca}	Autogenous shrinkage (assumed to be zero as restraint is internal)	$0\,\mu\varepsilon$	Section 3.2.2

f_E	Modulus of elasticity factor at 3 days	0.857	$f_E = \beta_{cc}(t)^{0.3}$ $= \exp\{s[1-(28/t)^{0.5}]\}^{0.3}$ Eq. 3.4 & 3.5 EN1992-1-1
E_{cm} (t_3)	Mean value of modulus of elasticity at 3 days	32.1 kN/mm^2	$E_{cm}(t_3) = E_{cm}.f_E$

Area of concrete in the tensile zone, A_{ct}. For internal restraint this is taken as 20% of the total concrete area.

A_{ct} $= 0.2 \cdot h \cdot b$ — Section A7.4.4

$= 0.2 \times 1{,}500 \times 1{,}000$

$= 300{,}000\,\text{mm}^2$

The effective area of concrete in tension around the reinforcement, $A_{c,eff}$, with depth limited to $h_{c,ef}$.

$h_{c,ef}$ is taken as the lesser of $h/2$ or $2.5 \times (c + \varphi_s /2)$ — Section 7.3.2 BS EN 1992-1-1

$h / 2 = 1{,}500/2 = 750\,\text{mm}$

$2.5 \cdot (c + \varphi_s /2) = 2.5 \times (50 + 16/2) = 145\,\text{mm}$

Therefore $h_{c,ef} = 145\,\text{mm}$

$A_{c,eff}$ $= b \cdot h_{c,ef} = 1{,}000 \times 145 = 145{,}000\,\text{mm}^2$

$\rho_{p,eff}$ Ratio of reinforcement to effective concrete area $A_{s,prov} / A_{c,eff}$ — Section 3.5.1

$= 1{,}005 / 145{,}000$

$= 0.00693$

2.5.14 Restrained Strain

The early age restrained strain, ε_r, is calculated as follows:

ε_r $= K_{c1} (\alpha_c \cdot \Delta T_1) R_1$ — Section 3.2.2 & Eqn. 3.4

$= 0.65 (12 \times 10^{-6} \text{ x } 56) \times 0.42$

$= 0.65 \times 0.00056 \times 0.42$

$= 0.000183\,\varepsilon$

$= 183\,\mu\varepsilon$

The tensile strain capacity, $\varepsilon_{ctu(ea)}$

$\varepsilon_{ctu(ea)}$ $= 1.08 \times (f_{ctm(3)}/ E_{cm}(t_3))$ — Section 4.9.2

$= 1.08 \times (1.92 / 32.1 \times 10^3)$

$= 0.000065\,\varepsilon$

$= 65\,\mu\varepsilon$

Therefore cracking is predicted as $\varepsilon_r > \varepsilon_{ctu(ea)}$.

The next step is to calculate the anticipated crack spacing and crack width to ensure it does not exceed the permitted value.

The strain induced by early age cracking, ε_{cr}:

ε_{cr} $= \varepsilon_r - 0.5 \cdot \varepsilon_{ctu(ea)}$

$= \{183 - (0.5 \text{ x } 65)\} \text{ x } 10^{-6}$

$= 0.000151\varepsilon$

$= 151\mu\varepsilon$

Section 3.3 & Eqn. 3.6

The minimum area of reinforcement required per face to control early age cracking, $A_{s,min}$:

$A_{s,min}$ $= k_c \cdot k \cdot A_{ct} \cdot f_{ct,r(3)} / f_{yk}$

$= 0.5 \times 1.0 \times 300{,}000 \times 1.34 / 500$

$= 201{,}000 / 500$

$= 402\,\text{mm}^2$ per face

16 mm bars at 200 mm centres provides 1,005 mm^2/m

Therefore OK.

Section 3.4 & Eqn. 3.20

Determine crack spacing, $S_{r,max}$:

$S_{r,max}$ $= 3.4 \cdot c_{nom} + 0.425 \cdot k_1 \cdot \varphi_s / \rho_{p,eff}$

$= (3.4 \times 50) + (0.425 \times 1.143 \times 16 / 0.00693)$

$= 170 + 1122$

$= 1{,}292\,\text{mm}$

Section 3.5 & Eqn. 3.21

Therefore the early age crack width, w_k:

w_k $= S_{r,max} \cdot \varepsilon_{cr}$

$= 1{,}292 \times (151 \times 10^{-6})$

$= 0.20\,\text{mm}$

w_k < 0.3 mm allowable *therefore OK.*

Section 3.6 & Eqn. 3.22

Reinforcement summary:

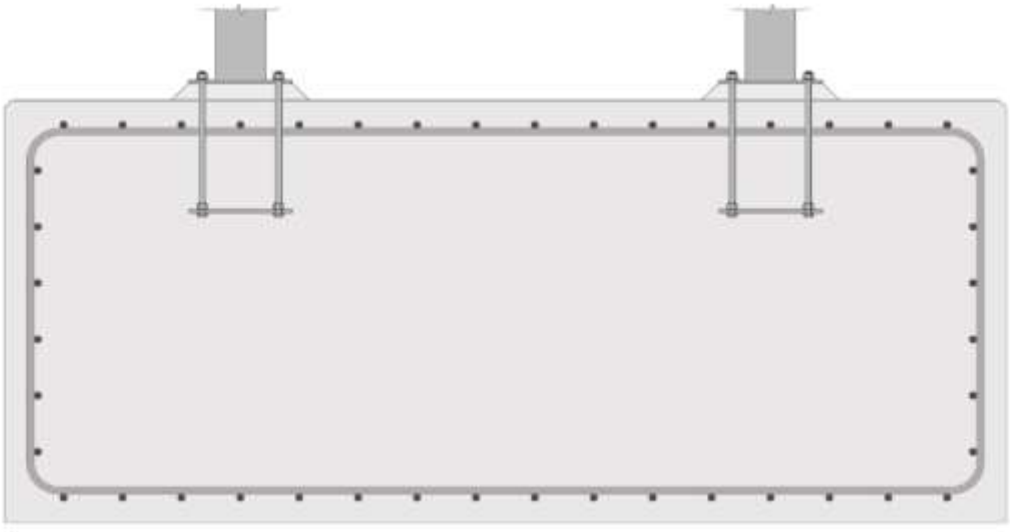
Reinforcement provision is adequate as the early age crack width < 0.3 mm.

The assumptions made to establish the thermal behaviour were based upon the most onerous conditions (e.g. concreting in summer and plywood rather than steel formwork). If the actual conditions during concreting are known and are less onerous then these should be used. This will result in reduced crack widths and may enable a reduction in reinforcement provision.

The reinforcement provision is governed in this example case by the limited crack width requirements rather than by flexural moments generated in the base by wind forces.

Therefore provide 16 mm bars at 200 mm spacing in both directions

Reinforcement (16 mm bars at 200 mm spacing) should also be provided to all other faces of the foundation to satisfy cracking requirements.



Indicative reinforcement arrangement. Laps and division of links are not shown for clarity, and should be determined considering the site-specific constraints and construction techniques. Note that the reinforcement arrangement shown is for a column with a flange plate.

2.5.15 Example concrete specification:

The concrete should be specified by a series of items that define its components and limiting values.

A typical specification for a sign base slab is as shown in the Table below.

Limiting values are based upon the requirements of BS 8500-1:2015 for the applicable assumed exposure classes. Minimum / maximum values are obtained from the most onerous exposure class requirement.

Note that if the sign foundation is exposed for example, to sea water spray, then more onerous conditions will apply.

Item	**Value**	*Reference BS 8500-1:2015 +A1:2016 unless denoted otherwise*
Intended Working Life of Structure (years)	25	*CD 354 Cl. 5.5*
Working life assumed in BS 8500-1 (years)	50	*Standard covers 50 or 100 years only*
Minimum specified cover to Reinforcement (mm) including an allowance for workmanship	50	*Table A.4*

Applicable Exposure Classes (XC, XD, XF)		
Corrosion induced by carbonation - *wet, rarely dry*	XC2	*Condition refers to reinforced concrete surfaces permanently in contact with soil not containing chlorides - Table A.1*
Corrosion induced by chlorides other than sea water - *cyclic wet and dry*	XD3	*Reinforced structure supports within 10m of a carriageway - Table A.1*
Freeze thaw attack - *moderate saturation*	XF1	*Concrete surfaces not highly saturated but exposed to freezing and to rain and water - Table A.1*
Minimum compressive Strength Class of concrete *(28 day cylinder, cube strength (N/mm^2)*	C35/45	*Assumed value in design – used to determine permissible cement types, maximum water to cement ratio and minimum cement or cement combination content*
Minimum Cement Content (kg/m^3)	380	*From Tables A4 & A9 - most onerous condition (XD3) governs*
Maximum Free Water/Cement Ratio	0.40	*From Tables A4 & A9 - most onerous condition (XD3) governs*
Required Type and Class of Cement or Combination		
Either: Portland cement with 21% to 35% fly ash	IIB-V	*Permissible cement types as described in Tables A.4 and A.6*
or Portland cement with 36% to 65% ggbs*	IIIA	
Maximum Cement Content (kg/m^3) * ggbs - Ground granulated blast furnace slag	550	*Highways Works Specification Series 1700 Clause 1704.3*

2.5.16 Conclusion

A foundation with $W = 3.40$ m, $L = 2.10$ m and $T = 1.50$ m is sufficient for the design.

A smaller foundation, particularly one with a reduced thickness, T, might also be suitable.

Refer to Example 1 for planted bases, which are often an economical alternative.

Example 2 Section 7: Points to consider

2.6.1 Passive safety, impact design, CE / UKCA marking and documentation

The points discussed for Example 1 above in paragraphs 1.8.1 and 1.8.2 are equally applicable to the specification of the sign in this example.

APPENDIX D: REFERENCES

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4. BS EN 1990:2002, Eurocode 0: Basis of Structural Design (with 2005 National Amendment).
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30. MCHW, Vol 1, Specification for Highway Works, Series 1700 - Structural Concrete

31. Free software for determining both the appropriate wind action and suitable steel support sections using the methods in this Guide, *SignLoad*, is available from: www.BuchananComputing.co.uk

32. Passive Safety UK Guidelines, lists of available passively safe products and other background information: www.passivesafetyuk.com or www.ukroads.org/passivesafety/publications

DMRB = Design Manual for Roads and Bridges, Highways England

MCHW = Manual of Contract Documents for Highway Works, Highways England

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