Impact on car park structures

Designing car park steelwork for impact has two aspects – the structural resistance and the selection of an appropriate steel sub-grade. David Brown of the SCI offers advice on both issues.

Advisory Desk 456 was prepared as a response to reports of designers circumnavigating the requirement to design internal columns for impact. Without turning to any design standard it seems entirely to be expected that accidents will happen within car parks leading to impact on any unprotected structural elements. There is plenty of evidence in car parks that vehicles can and do hit walls and barriers – regularly.

BS EN 1991-1-1 Annex B covers "Vehicle barriers and parapets for car parks" and gives a method for calculating the horizontal characteristic force on a barrier from vehicle impact. Logic demands that if a vehicle can hit a barrier, it can equally hit a column, if unprotected, so this Annex can be used to calculate the force applied to any unprotected element.

It seems that some designers are looking at the UK National Annex to BS EN 1991-1-7, and in particular at clause NA.2.16. In clause NA.2.16 the NA states that the equivalent static design force due to vehicular impact should be taken from Table 4.2 of BS EN 1991-1-7, unless the structure is Consequence Class 3. If the structure is Consequence Class 3, the National Annex directs the designer to Table NA.9.

Table 4.2 of BS EN 1991-1-7 includes traffic categories of motorways, main roads, country roads, courtyards and parking garages. It is absolutely clear that this table refers to impact on a structure from the outside – motorway velocity is not anticipated inside a multi-storey car park.

Car park structures become Consequence Class 3 if they have more than 6 storeys, when Table NA.9 applies. Table NA.9 also has classes of road including motorway, trunk roads etc – it is equally clear that this table applies to impact from outside a building. There can be no mistake – the note to the table states categorically that "these equivalent design forces are applicable outside a building; for columns inside any multi-storey building used for car parking the value must be taken from BS EN 1991-1-1 Annex B".

It is reported that for Consequence Class 2 car park structures (those not exceeding 6 storeys), some designers suggest that they are not required to look at Table NA.9, and therefore avoid the note that internal columns should be designed for the forces in BS EN 1991-1-1 Annex B. This thought process does leave an unanswered question as to what forces should then be used. It seems that in this situation, designers are using the forces associated with external impact, from Table 4.2 of BS EN 1991-1-7, ignoring that fact that it is not applicable for impact internally and ignoring the inconvenient logic that if a vehicle can hit a barrier, it can also hit an unprotected internal column. The attraction of this thought process is that Table 4.2 of BS EN 1991-1-7 specifies a mere 75 kN for columns in "courtyards and parking garages", in contrast to the higher forces determined from Annex B of BS EN 1991-1-1.

Annex B of BS EN 1991-1-1 should be used to determine the impact forces on unprotected elements within a multi-storey car park, whatever their Consequence Class.

Impact force according to Annex B of BS EN 1991-1-1

The characteristic impact force F is given by:

$F = 0.5mv^2 / (\delta_c + \delta_b)$

where:

m is the mass of the vehicle in kg, taken as 1500 kg for vehicles with a gross mass not exceeding 2500 kg

v is the velocity of the vehicle in m/s, taken to be 4.5 m/s

δ_c is the deformation of the vehicle, taken to be 100 mm

δ_b is the deformation of the barrier (or in this case, the column)

Clearly δ_b is a function of the member stiffness and the applied load, so some iteration is needed to find the force F.

Considering a column supporting four storeys above, the ultimate axial load $N_{b,Ed}$ is approximately 3850 kN. This value is based on a column grid of 7.2 m × 15.6 m, a variable action of 2.5 kN/m² and a permanent action of 3.56 kN/m². If the storey height is 3.5 m, a 305 UC 118 in S355 would be appropriate, with $N_{R,Ed} \approx 4115$ kN.

As a first guess, the deformation δ_b has been taken as 5 mm.

Then $F = 0.5 \times 1500 \times 4.5^2 / (100 + 5) = 145$ kN

Annex B specifies that the load is applied at 375 mm above floor level, so the column has been analysed as shown in Figure 1, with the load applied 825 mm from the node – based on half a 600 mm deep beam and a 150 mm slab, plus 375 mm. The ends of the analysis member have been modelled as fixed, which is considered to be a reasonable assumption for continuous columns and the sudden application of the load.

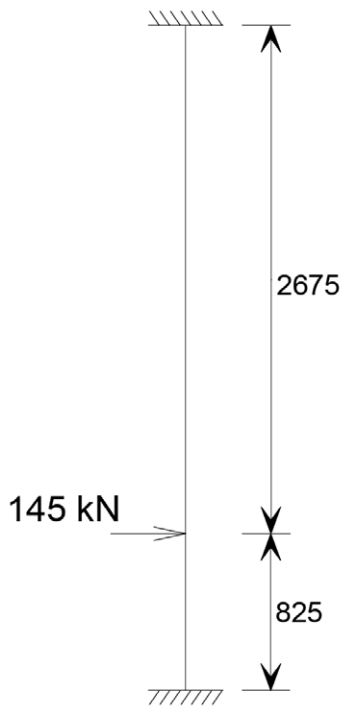


Figure 1: Analysis model

To make life easy, the member was modelled with a node at the point of load application, so the deflection could be extracted readily from the analysis results. Two cases must be considered – load applied to either axis.

In the major axis, under a load of 145 kN, the deflection at the point of load application is 0.21 mm.

The force F_{major} is revised to $0.5 \times 1500 \times 4.5^2 / (100 + 0.21) = 152$ kN.

Under this load, the deflection does not change significantly, so the characteristic force F is taken as 152 kN.

In the minor axis, under a load of 145 kN, the deflection at the point of load application is 0.64 mm

The force F_{minor} is revised to $0.5 \times 1500 \times 4.5^2 / (100 + 0.64) = 151$ kN.

Under this load, the deflection does not change significantly, so the characteristic force is taken as 151 kN

The bending moment due to this load is 73.2 kNm at the adjacent support (72.8 kNm in the minor axis)

Column verification

The impact on the column is an accidental situation (despite the frequency one sees damage in a car park) and therefore the column is verified under a combination of actions according to equation 6.11b of BS EN 1990.

The characteristic axial load from the permanent actions is 1600 kN, and from the variable actions is 1120 kN, leading to a total of 2720 kN.

From the UK NA to BS EN 1990, Table NA.A1.1 gives ψ_1 as 0.7

Thus the design combination axial load is $1600 + 0.7 \times 1120 = 2384$ kN

The accidental action is unfactored in equation 6.11b, so the design bending moment is 73.2 kNm in the major axis.

The column should be verified in combined bending and axial compression, using expressions 6.61 and 6.62 of BS EN 1993-1-1. This involves laborious determination of the interaction factors if proceeding with manual calculations, so to use a spreadsheet or other software would be a wise decision at this point.

Thankfully, there is a convenient software for combined axial compression and bending available on *steelconstruction.info*. Entering the input parameters, the results are shown in Figure 2 (over page) for the major axis.

The complication of calculating the C_1 factor was avoided by setting the moment at both ends to be 73.2 kNm. A uniform moment is the most onerous, so the approach is conservative. With both resulting utilization factors less than 1.0, the column is satisfactory.

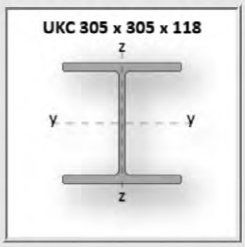
The results for the minor axis are shown in Figure 3 (over page).

Impact and steel sub-grade

When specifying a steel sub-grade, designers will refer to BS EN 1993-1-10 and the UK National Annex. Hopefully, they will use PD 6695-1-10 as a much easier approach if fatigue is a design consideration, and SCI publication P419 if fatigue is not a concern. However, these resources only allow for a modest strain rate ▸28

Output

Section	UKC 305 x 305 x 118				
Buckling Length	3.5 m				
Steel Grade	S355				
Design Strength	345 N/mm^2				
Axial Compression	2348 kN				



Major axis moment varies linearly from 73.2 kNm to 73.2 kNm

Minor axis moment varies linearly from 0 kNm to 0 kNm

$M_{b,y,Rd}$	= 648 kNm	$C_{m,LT}$	= 1.00	$k_{y,y}$	= 1.060
$M_{c,z,Rd}$	= 309 kNm	$C_{m,y}$	= 1.00	$k_{z,y}$	= 0.956
$N_{b,y,Rd}$	= 4930 kN	$C_{m,z}$	= 0.60	$k_{y,z}$	= 0.475
$N_{b,z,Rd}$	= 4120 kN			$k_{z,z}$	= 0.792

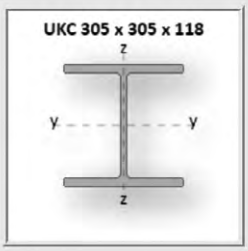
Expression 6.61 = 0.597

Expression 6.62 = 0.678

Figure 2: Column verification – major axis moment

Output

Section	UKC 305 x 305 x 118				
Buckling Length	3.5 m				
Steel Grade	S355				
Design Strength	345 N/mm^2				
Axial Compression	2348 kN				



Major axis moment varies linearly from 0 kNm to 0 kNm

Minor axis moment varies linearly from 72.8 kNm to 72.8 kNm

$M_{b,y,Rd}$	= 675 kNm	$C_{m,LT}$	= 0.60	$k_{y,y}$	= 0.638
$M_{c,z,Rd}$	= 309 kNm	$C_{m,y}$	= 0.60	$k_{z,y}$	= 0.905
$N_{b,y,Rd}$	= 4930 kN	$C_{m,z}$	= 1.00	$k_{y,z}$	= 0.792
$N_{b,z,Rd}$	= 4120 kN			$k_{z,z}$	= 1.320

Expression 6.61 = 0.663

Expression 6.62 = 0.88

Figure 3: Column verification – minor axis moment

▸27

covering most transient and persistent design situations. In clause 2.3.1(2), BS EN 1993-1-10 notes that for other strain rates (e.g. impact loads), the tabulated values must be modified.

The first challenge is to determine the strain rate, so the equations of motion once learned in physics lessons finally have some use. The mass is known, and the force, so the acceleration can be determined from $F = m.a$

Knowing the acceleration, the initial and final velocities, the time can be determined from $v = u + a.t$

With some trivial rearrangement, $t = \frac{um}{F} = \frac{4.5 \times 1500}{152 \times 1000} = 0.044$ seconds.

The maximum stress due to the impact is when the force is applied in the minor axis.

The stress is $\frac{M}{W_{z,el}} = \frac{72.8 \times 10^6}{598 \times 10^3} = 123$ N/mm^2

The strain is therefore $\frac{123.6}{210000} = 5.89 \times 10^{-4}$

and the strain rate $= \frac{5.89 \times 10^{-4}}{0.044} = 0.0134$/sec

Due to this high strain rate, the reference temperature in Table 2.1 of BS EN 1993-1-10 must be reduced by $\Delta T_{\dot{\varepsilon}}$ given by:

$$\Delta T_{\dot{\varepsilon}} = -\frac{1440 - f_y(t)}{550} \times \left(\ln \frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right)^{1.5} = -\frac{1440 - 345}{350} \times \left(\ln \frac{0.0134}{1 \times 10^{-4}}\right)^{1.5} = -21.6°$$

It should be noted that in the above expression, $\dot{\varepsilon}_0$ has been taken as 1×10^{-4}/sec. This is not at all clear in clause 2.3.1 of BS EN 1993-1-10 where the value of $\dot{\varepsilon}_0 = 4 \times 10^{-4}$/sec appears. Designers would be forgiven for using this latter value, but this is the value allowed for in the tabulated values, not the value to be used to calculate $\Delta T_{\dot{\varepsilon}}$. Although not given in the code, the use of $\dot{\varepsilon}_0 = 1 \times 10^{-4}$/sec is explained in Reference 1, and confirmed in the draft prEN 1993-1-10.

The steel sub-grade may now be determined. Because of the temperature shift, immediate use of the final tables in either PD 6695-1-10 or P419 is not possible. The following example demonstrates the application of the UK National Annex provisions, assuming fatigue is not a design consideration, and therefore using information from P419. The NA references are to the UK NA to BS EN 1993-1-10.

Firstly, NA.2.1.2.2 specifies that in the UK, the use of Table 2.1 in the Eurocode is limited – only the section for $\sigma_{Ed} = 0.75 f_y(t)$ may be used. P419 presents data for an extended range of reference temperatures in Table 4.1 for $\sigma_{Ed} = 0.75 f_y(t)$, which will be used in this example.

It is assumed that the steelwork is external, so the service temperature T_{md} is -15°C.

It is assumed that the steel is welded generally, with no particular onerous details. From NA.2.1.1.2, $\Delta T_{RD} = 0$°C

It is assumed that there are no stress concentrations, so from NA.2.1.1.3, $\Delta T_{Rg} = 0$°C

If the steel is JR sub-grade, the test temperature is room temperature, 20°C. According to table NA.3, the difference between the test temperature and the service temperature is 20 – (-15) = 35°C and the value of $\Delta T_{RT} = -30$°C

Under the previous calculated axial load of 2348 kN and the moment of 72.8 kNm, the cross section is all in compression, so according to Table NA.5, $\Delta T_{R\sigma} = 30$°C

Allowing for the adjustment $\Delta T_{\dot{\varepsilon}} = -21.6$°C, the reference temperature becomes:

$T_{Ed} = -15 - 21.6 - 30 + 30 = -36.6$°C

An extract of Table 4.1 from P419 is shown in Figure 4.

Steel Grade	Sub-Grade	Charpy energy CVN at T (°C)	Charpy energy CVN J_{min}	Reference temperature T_{Ed} (°C) $\sigma_{Ed} = 0.75 f_y(t)$												
				70	60	50	40	30	20	10	0	–10	–20	–30	–40	–50
	JR	20	27	200	200	200	200	200	200	200	177	114	77	54	40	30
	J0	0	27	200	200	200	200	200	200	200	200	200	177	114	77	54
S355	J2	–20	27	200	200	200	200	200	200	200	200	200	200	200	177	114

Figure 4: Extract from Table 4.1, P419

From Figure 4, the limiting thickness even at -40°C is 40 mm, compared to the actual flange thickness of 18.7 mm, so JR is satisfactory.

A design case could be considered where the vehicle strikes the column in an otherwise empty car park. The axial load is then reduced to 1600 kN. Under this load, the compression is 106 N/mm², so there is a net tension of 17.6 N/mm² on the extreme fibres.

Therefore, 17.6/345=0.05 and from Table NA.5, $\Delta T_{\dot{\varepsilon}} = 20$°C.

$T_{Ed} = -15 - 21.6 - 30 + 20 = -46.6$°C

At -50°C, the limiting thickness is 30 mm, still more than the flange of 18.7 mm and JR remains satisfactory. ■

1 Sedlacek, G, et al
Commentary and worked examples to EN 1993-1-10 "material toughness and through thickness properties" and other toughness oriented rules in EN 1993
Joint Research Centre, 2008