ZUSSMAN BEAR PARTNERSHIP
DATE: NOVEMBER 1998

ZB

PS9805005 R1
LS9805006 R1

STRUCTURAL DESIGN CALCULATIONS

FOR

25-27 SICILIAN AVENUE
HOLBORN
LONDON WC1

DESIGN DATA

ALL DESIGN IS CARRIED OUT IN ACCORDANCE WITH THE FOLLOWING BRITISH STANDARDS AND CODES OF PRACTICE

BS 6399	DESIGN LOADING FOR BUILDINGS
BS 648	WEIGTH OF BUILDING MATERIALS
BS 5628	STRUCTURAL USE OF MASONRY
BS 449	STRUCTURAL USE OF STEEL

FIRE REQUIREMENTS	½ HOUR
TIMBER BENDING STRESS	5.3 N/mm2
COMPRESSIVE STRESS OF EXISTING MASONRY	0.42 N/mm2

LOADING DATA

WALL LOADING

Existing 9" wall plastered one side	5.1 kN/m2
Existing stud wall brick infill	2.7 kN/m2
Existing 13.5" wall plastered both sides	8.0 kN/m2
Existing stud wall	0.7 kN/m2
Single plain brick	2.3 kN/m2
Glass	0.16 kN/m2

LOADING DATA

FLOOR LOADING

BOARDING	0.13
FLOOR JOISTS	0.16
INSULATION	0.05
CEILING	0.12
75mm SCREED	1.8
TOTAL (DL)	2.3
IMPOSED LOAD DOMESTIC	2.5
TOTAL LOADING	**4.76 kN/m2**

ZUSSMAN BEAR PARTNERSHIP	SHEET No. 61
St. John's Studios, Richmond, TW9 2QA Tel: 0181-332 1199 Fax: 0181-332 9199	BY ______ DATE NOV 9
	ITEM
TITLE 25-27 Sicilian Ave. NO 4/7198	DRAWING Nos

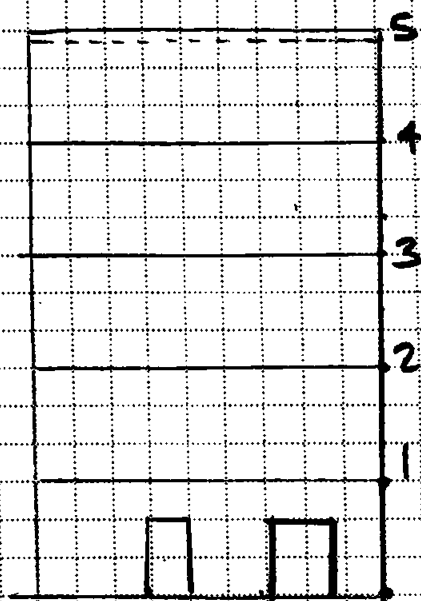
LOCATION	CALCULATION	OUTPUT

Opening clear width = 1.25m.
to ℄ of columns.

1.35 say.

wall thickness say 330

5 storey load.



5 0.86 x 3.5 = 3.01 kN/m
0.75 x 3.5 = 2.6 kN/m.

4 2.3 x 3.5 = 7.7 kN/m. (DL)
2.5 x 3.5 = 8.7 kN/m. (LL)

3 7.7 kN/m
8.7 kN/m.

2 7.7 kN/m
8.7 kN/m

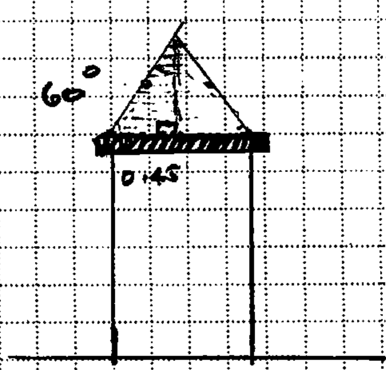
1 7.7 kN/m
8.7 kN/m

Total load from roof & floor. = (DL) = 34 kN/m.
(LL) = 38 kN/m

Wall self. = (DL) = 15 x 8 = 120 kN/m.

∴ Total (DL) = 154 kN/m.
(LL) = 38 kN/m.

Lintel above Door, span = 0.9m.



Area Approx $1 \times 0.35 \times 2 = 0.7$ say $1.0m^2$

∴ loading = 15.0 kN/m.

USE 3 Number. 150 X 100

Capacity = $35.6 \times 3 = 106.8$ kN > 15.0 kN OK

Boxframe for 1.5m opening

Assume full loading.

$(154+38) = 192$ kN

$BM = 192 \times 1.5^2/8 = 54$ kNm.

$R = 192 \times 1.5/2 = 144$ KN.

Go for Double frame.

∴ $BM = 192/2 = 96 \times 1.5^2/8 = 27$ kNm.

$R = 96 \times 1.5/2 = 72$ kN. Per frame.

ZUSSMAN BEAR PARTNERSHIP

St. John's Studios, Richmond, TW9 2QA Tel: 0181-332 1199 Fax: 0181-332 9199

SHEET No. 03
BY DATE Nov98
ITEM
DRAWING Nos

TITLE 25-27 Sicilian Ave. NO 4/7198

LOCATION	CALCULATION	OUTPUT

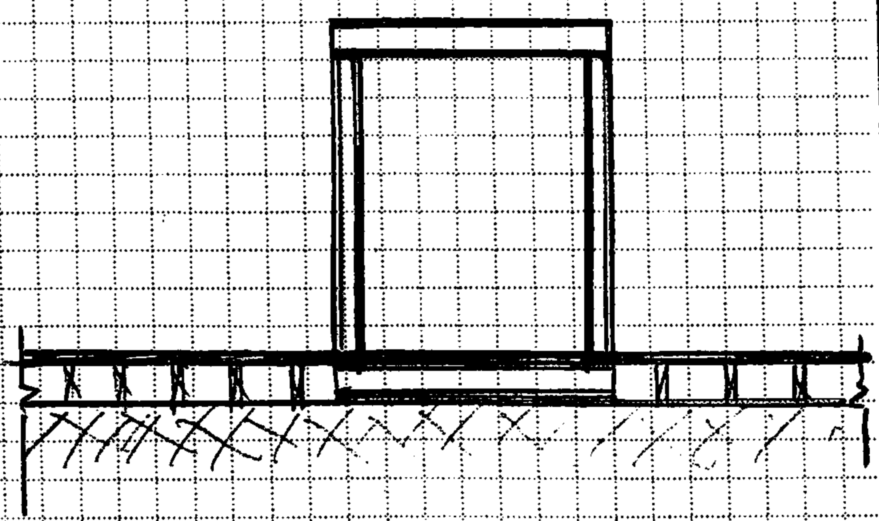
Double Boxframe.

(DL) = 154/2 = 77 kN/m.
(LL) = 36/2 = 19 kN/m

Try & use Double 152x152x23 UC

See Computer calc.
Page 1 - 4

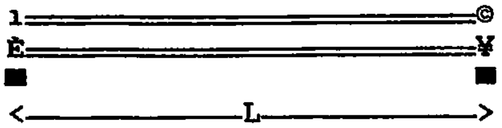
Columns. Reaction = 72 kN.



Floor Depth.
Say 200.

Location: BOX FRAME OPENING

Simply supported steel beam



Beam span	L=1.5 m
Young's modulus	E=205 kN/mm2

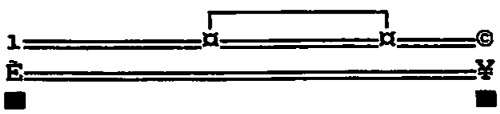
Section properties

Section size	152 x 152 x 23 Universal Column
Depth of steel section	D=152.4 mm
Width of steel section	B=152.4 mm
Thickness of web	t=6.1 mm
Thickness of flange	T=6.8 mm
Root radius	r=7.6 mm
Inertia about major axis	Ix=1260 cm4
Inertia about minor axis	Iy=403 cm4
Plastic modulus about major axis	Sx=184 cm3
Torsional constant	J=4.87 cm3
Area of section	A=29.8 cm2
Radius gyration about minor axis	ry=SQR(Iy/A) =SQR(403/29.8) =3.6774 cm

Dead load factor	gamd=1.4
Imposed load factor	gami=1.6

All loads are positive downwards, reactions are positive upwards, sagging moments are positive.

<———Lbu———> Gku Characteristic dead UDL
<—Lau—> Qku Characteristic imposed UDL



Distances are measured from left hand support

Uniformly distributed load 1 of 1

Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=1.5 m
Characteristic dead load	Gku(1)=77 kN/m (unfactored)
Characteristic imposed load	Qku(1)=19 kN/m (unfactored)

BMs at 20th points, from left to right (sagging is positive)

0	7.3851	13.993	19.823	24.876	29.152
	32.65	35.371	37.314	38.48	38.869
	38.48	37.314	35.371	32.65	29.152
	24.876	19.823	13.993	7.3851	0

Maximum span bending moment 38.869 kNm

End shears

Shear force at left hand end	103.65 kN
Shear force at right hand end	103.65 kN
Design shear force	Fv=Rlhe
	=103.65 kN

Unfactored dead shear at LHE	57.75 kN
Unfactored imposed shear at LHE	14.25 kN
Unfactored dead shear at RHE	57.75 kN
Unfactored imposed shear at RHE	14.25 kN

Midspan deflections

Unfactored dead load deflection	1.965 mm
Unfactored imposed load deflectn	0.48488 mm
Total DL & imposed deflection	2.4499 mm
Span:defln ratio for dead load	763.35
Span:defln ratio for imposed load	3093.6
Span:defln ratio for total load	612.27
Limiting deflection	DELlim=L*1000/360
	=1.5*1000/360
	=4.1667 mm

Since imposed load deflection <= DELlim (0.48488 <=4.1667)
deflection within limiting value.

Strength of steel Clause 3.1.1

For thickness of 6.8 mm

Design strength (Grade 43)	py=275 N/mm2

Section classification

Constant (Table 7 Note 3)	e=(275/py)^0.5
	=(275/275)^0.5
	=1
Outstand	b=B/2
	=152.4/2
	=76.2 mm
Ratio	b'T=b/T
	=76.2/6.8
	=11.206
Compact limiting value of ratio	b'Tlim=9.5*e
	=9.5*1
	=9.5

b/T ratio exceeds limiting value of 9.5e.

Semi-compact limiting value	b'Tls=15*e
	=15*1
	=15

b/T ratio within semi-compact limiting value of 15e.

Compact limiting value of ratio	d'tlim=98*e
	=98*1
	=98
Depth between fillets	d=D-2*(T+r)
	=152.4-2*(6.8+7.6)
	=123.6 mm
Ratio	d't=d/t
	=123.6/6.1
	=20.262

d/t ratio within limiting value of 98e.
Therefore section is semi-compact.

Buckling resistance

Since the beam is subject to possible lateral torsional buckling, the buckling resistance moment Mb is first considered rather than the moment capacity Mc as a guide to selection. A conservative analysis is adopted, taking the worst moment with the greatest effective length.

```
Equivalent uniform moment          Mbar=M
                                       =38.869 kNm
Greatest effective length          Le=1.5 m
Slenderness of section             lambda=(Le*1000)/(ry*10)
                                         =(1.5*1000)/(3.6774*10)
                                         =40.789
Torsional index                    x=0.566*((D-T)/10)*(A/J)^0.5
                                    =0.566*((152.4-6.8)/10)*(29.8
                                     /4.87)^0.5
                                    =20.386
Ratio                              ratio=lambda/x
                                        =40.789/20.386
                                        =2.0009
Factor                             N=0.5
Slenderness factor                 v=TABLE 14 for ratio=2.0009, N=0.5
                                    =0.95995
Factor from Table 13               n=1.0
Appendix B.2 is used to calculate the buckling resistance moment.
Buckling parameter                 u=(4*Sx^2*(1-Iy/Ix)/(A^2*((D-T)
                                     /10)^2))^0.25
                                    =(4*184^2*(1-403/1260)/(29.8^2
                                     *((152.4-6.8)/10)^2))^0.25
                                    =0.83635
Equivalent slenderness             lamlt=n*u*v*lambda
                                        =1*0.83635*0.95995*40.789
                                        =32.748
Limiting slenderness               lamlo=0.4*((PI^2*E*10^3)/py)^0.5
                                        =0.4*((3.1416^2*205*10^3)
                                         /275)^0.5
                                        =34.31
Perry coefficient                  etaLT=0.007*(lamlt-lamlo)
                                        =0.007*(32.748-34.31)
                                        =-0.010934
Limitimg value of coefficent       etaLT=0
Plastic moment capacity            Mp=Sx*py/10^3
                                     =184*275/10^3
                                     =50.6 kNm
Elastic critical moment          Me=(Mp*PI^2*E*10^3)/(lamlt^2*py)
                                   =(50.6*3.1416^2*205*10^3)/(32.748^2
                                    *275)
                                   =347.14 kNm
Factor                             phiB=(Mp+(etaLT+1)*Me)/2
                                       =(50.6+(0+1)*347.14)/2
                                       =198.87 kNm
Buckling resistance moment  Mb=(Me*Mp)/(phiB+(phiB^2-Me*Mp)^0.5)
                              =(347.14*50.6)/(198.87+(198.87^2-347.14
                               *50.6)^0.5)
                              =50.6 kNm
```

Since Mbar<Mb (38.869 < 50.6) section O.K. for lateral torsional buckling resistance.

Shear capacity

Shear area	Avx=D*t
	=152.4*6.1
	=929.64 mm2
Shear capacity	Pv=0.6*py*Avx/1000
	=0.6*275*929.64/1000
	=153.39 kN

Since Fv <= Pv (103.65 <=153.39) shear force in web within shear capacity.

Moment capacity

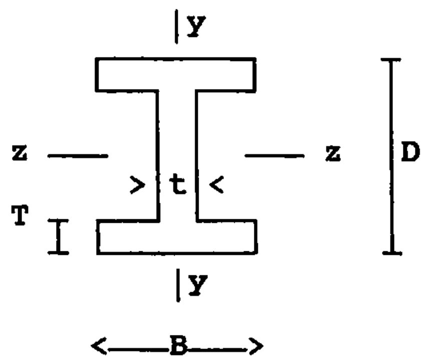
Elastic modulus	Zx=Ix/(D/20)
	=1260/(152.4/20)
	=165.35 cm3
Mt. capacity for semi-compact sec	Mc=py*Zx/1000
	=275*165.35/1000
	=45.472 kNm

Since M <= Mc (38.869 <=45.472) design moment within moment capacity.
N.B. Moment capacity is less than buckling resistance and therefore moment capacity controls the design.

UNIVERSAL COLUMN	152 x 152 x 23 UC	Grade 43
	Design shear force	103.65 kN
DESIGN	Shear capacity	153.39 kN
SUMMARY	Design moment	38.869 kNm
	Moment capacity	45.472 kNm
	Buckling resistance	50.6 kNm
	Deflection due to IL	0.48488 mm
	Limiting deflection	4.1667 mm

Office: 1160

Location: BOX FRAME COLUMN



I section column

Calculations in accordance with BS449 : Part 2 : 1969

Design BM about major axis zz	Mzz'=0 kNm
Design shear force in y direction	SF'=0 kN
Design axial load	F'=72 kN
Effective length about zz axis	lzz'=2.5 m
Depth of section	D=127 mm
Width of section	B=76.2 mm
Thickness of flange	T=7.6 mm
Thickness of web	t=4.5 mm
Design BM about minor axis yy	Myy'=0 kNm
Effective length about yy axis	lyy'=2.5 m
Steel Grade	Grade=43

Section properties

Constant
b=B-t
=76.2-4.5
=71.7

Constant
d=D-2*T
=127-2*7.6
=111.8

Take root radius for sect props
r=t
=4.5 mm

Area of section
area=B*D-b*d+0.2146*r^2*4
=76.2*127-71.7*111.8+0.2146 *4.5^2*4
=1678.7 mm2

Inertia of section
Izz=(B*D^3-b*d^3)/12+0.2146*r^2*4*(D/2-T-0.233*r)^2
=(76.2*127^3-71.7*111.8^3)/12+0.2146*4.5^2*4*(127/2-7.6-0.233*4.5)^2
=4709976 mm4

Radius of gyration
rzz=SQR(Izz/area)
=SQR(4709976/1678.7)
=52.969 mm

Inertia of section
Iyy=(2*T*B^3+d*t^3)/12
=(2*7.6*76.2^3+111.8*4.5^3)/12
=561287 mm4

Radius of gyration
ryy=SQR(Iyy/area)
=SQR(561287/1678.7)
=18.285 mm

Slenderness ratio
l'rz=lzz/rzz
=2500/52.969
=47.198

Slenderness ratio
l'ry=lyy/ryy
=2500/18.285
=136.72

Controlling slenderness ratio   l'r=l'ry
=136.72

D/T ratio   D'T=D/T
=127/7.6
=16.711

Axial compression

Calculated compressive stress   fc=F/area
=72000/1678.7
=42.89 N/mm2

Allowable compressive stress   pc=TABLE 17 for l'r=136.72, Grade=43
=49.295 N/mm2

Since fc <= pc (42.89 <=49.295) compressive stress within that given in BS449 Table 17, therefore OK.