INTERNATIONAL UNIVERSITY OF AFRICA

CIVIL ENGINEERING DEPARTMENT

ANALYSIS AND DESIGN OF STEEL WORKS II

GRADE 4

8TH SEMESTER

Lecture No 5

PLATE GIRDERS

Part 1

PLATE GIRDERS

1 Introduction

The high bending moments and shear forces associated with carrying large loads over long spans will frequently exceed the capacity of universal beam sections. In this situation, plate girders may be fabricated, their proportions being designed to provide a high strength to weight ratio.

In a fabricated plate girder,

1. The primary function of the flanges is to resist axial compressive and tensile forces arising from the bending moments.
2. The primary function of the web is to resist the shear force.

For an efficient plate girder design, the web depth ***d*** should be increased as far as possible to give the lowest flange force for a given bending moment. To reduce self-weight, the web thickness ***t*** should be reduced to a minimum. The consequence of these requirements is that the web has a high ***d/t*** ratio and tends to buckle in shear if stiffeners are not provided.

For an economic design, advantage should be taken of the post buckling reserve of strength commonly known as *"tension field action"*. BS 5950-1 does allow this reserve of strength to be taken into account. It is inevitable that the increased efficiency of designs to BS 5950-1 leads to some additional complexity of design calculations. There are special requirements for the ends of the plate girders in order to anchor the "tension field action". A typical plate girder is shown in Figure.1. انظر الترجمـــــــــــــه

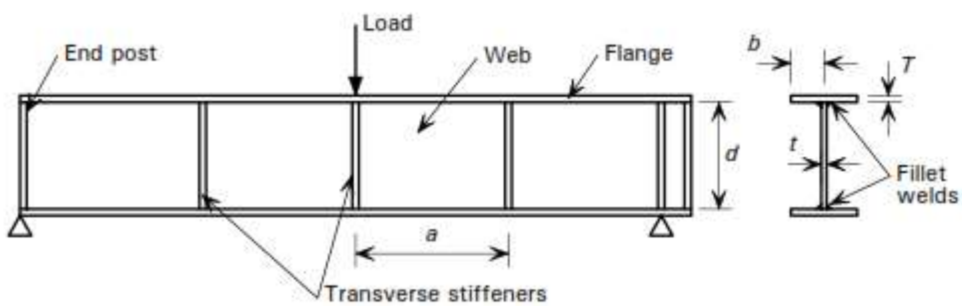


Figure .1 Typical plate girder with transverse stiffeners

The designer has to make the following decisions:

1. *Depth of girder (Generally, span/8 to span/16 is a reasonable depth range, where no other restrictions exist.)*
2. *Size of flange plates*
3. *Web thickness*
4. *Stiffener spacing*

1- DEPTH OF PLATE GIRDER

SPAN /8 TO SPAN /16

USE SPAN/10

2-SIZE OF FALANGE PLATE

$$A_f = \frac{M}{D p_y}$$

3-WEB THICKNESS

$$t = \frac{D}{150}$$

where

L = span of beam

A_f= area of flange

M = maximum ultimate bending moment

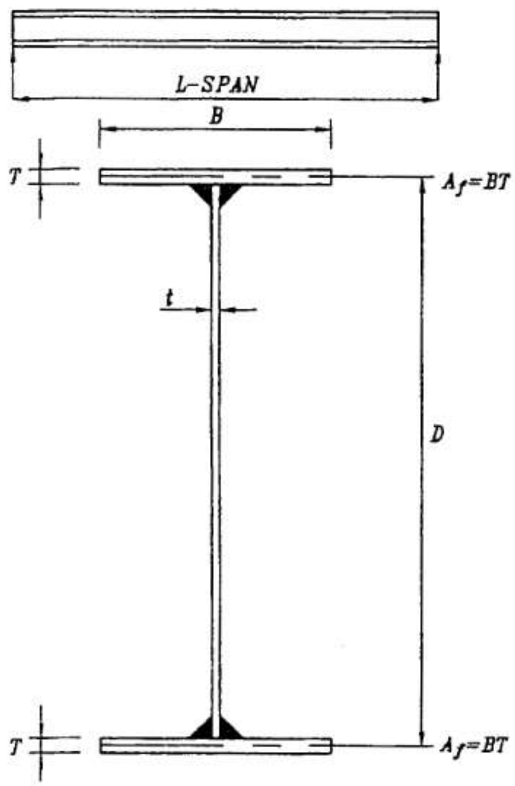


Figure 2 plate girder designation

The minimum web thickness t_{min} should further be checked as follows

(1) Minimum web thickness for serviceability:

− *where the stiffener spacing* $a > d$ $t \geq d/250$

− *where the stiffener spacing* $a \leq d$ $t \geq (d/250)(a/d)^{0.5}$

(2) Minimum web thickness to avoid compression flange buckling into the web:

- *where the stiffener spacing* $a > 1.5d$ $t = (d/250)(p_{yf}/345)$

where the stiffener spacing $a \leq 1.5d$ $t = (d/250)(p_{yf}/445)^{0.5}$

pyf *is the design strength of the compression flange*

In practice, these rules allow the use of very thin webs and impose little restriction on the design of plate girders.
IN THE ABOVE THE
THE COMPRESSION ﬂANGE IS FULLY RESTRAINED LATERALLY.

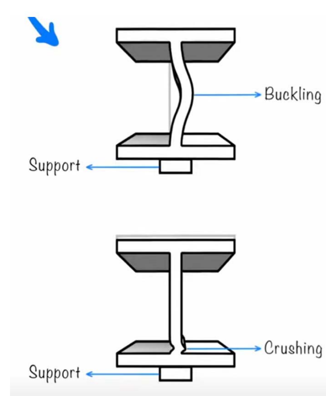
Numerical example

A simply supported plate girder has a span of 12 m and carries two concentrated loads on the top flange at the third points consisting of 450 kN dead load and 300 kN imposed load. In addition, it carries a uniformly distributed dead load of 20 kN/m, which includes an allowance for self-weight and an imposed load of 10 kN/m. The compression flange is fully restrained laterally. The girder is supported on a heavy stiffened bracket at each end. The material is Grade S275 steel. Design the girder using the simplified method for web first.

Solution

Step 1 Determine design strength

	Grade of steel	Thickness (mm)	Value of p_y (N/mm^2)
S275	43	≤16	275
		≤40	265
		≤63	255
		≤100	245
S355	50	≤16	355
		≤40	345
		≤63	340
		≤100	325
S420	55	≤16	450
		≤25	430
		≤40	415
		≤63	400



Step 2 Carry out analysis

The factored loads are:

Concentrated loads = $(1.4 \times 450) + (1.6 \times 300) = 1110\,\text{kN}$
Distributed load = $(1.4 \times 20) + (1.6 \times 10) = 44\,\text{kN/m}$

The loads and reactions are shown in Figure 2(a) and the shear force diagram in Figure 2 (b). The moments are: in Figure 2 (c)

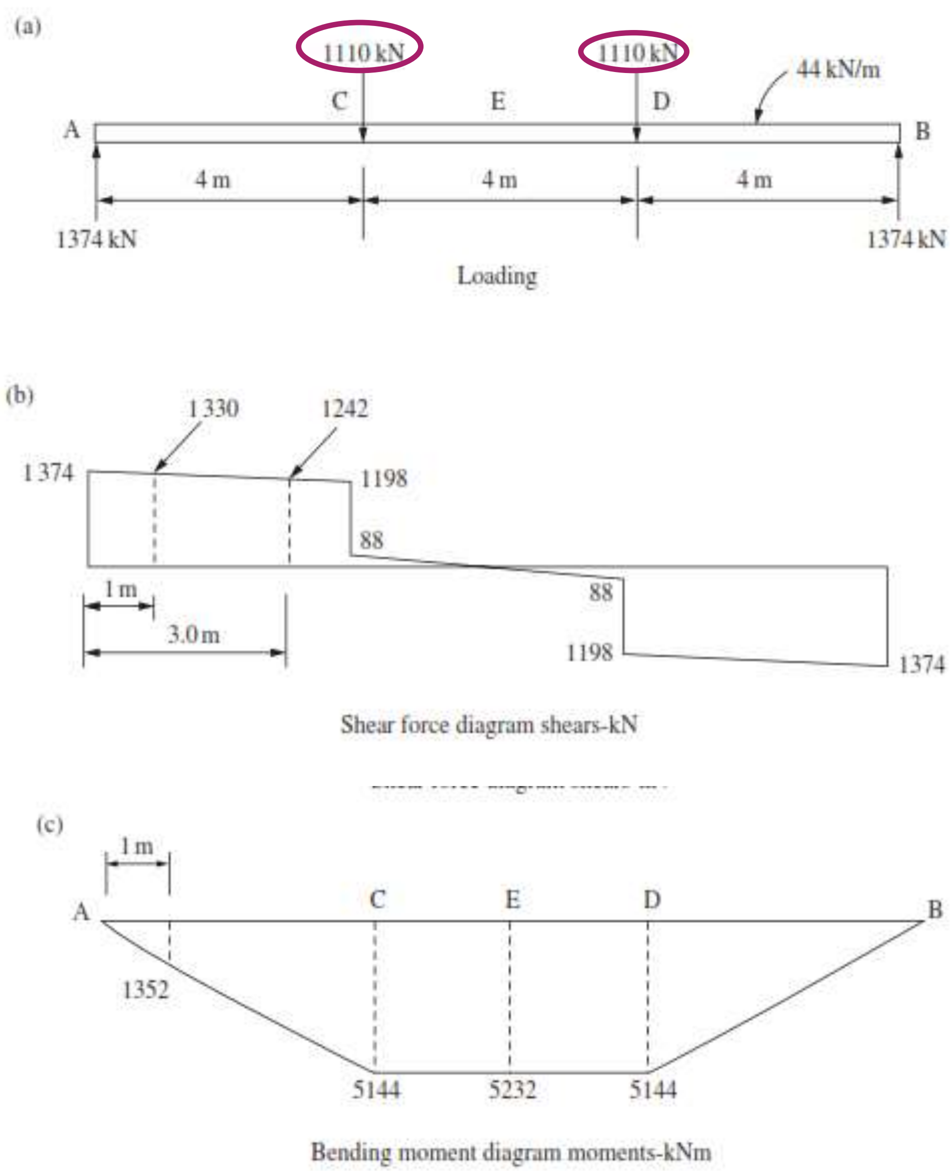


Figure 3 Load, shear and moment diagrams

Step 3 Determine internal forces

Determine bending moment for relevant load combinations. Deter-mine shear force for relevant load combinations. Find maximum co acting direct loads with bending moments.

$$M_C = (1374 \times 4) - (44 \times 4 \times 2) = 5144\,\text{kNm}$$
$$M_E = (1374 \times 6) - (1110 \times 2) - (44 \times 6 \times 3) = 5232\,\text{kNm}$$

Step 4 Select trial section and carry out section classification

(1) Design for girder depth span/10

Take the overall depth of the girder as 1200 mm and assume that the flange plates are over 40 mm thick. Then the design strength from BS 5950: Part 1, Table 9 for plates is py= 255 N/mm²

The flanges resist all the moment by a couple with lever arm of, say, 1140 mm, as shown in Figure 5.16(a). The flange area is:

$$A_f = \frac{M}{Dp_y}$$

$$A_f = \frac{5232 \times 10^6}{1200 \times 255} = 17098mm2$$

Make the flange plates 450 × 45 mm, giving an area of 20 250 mm2

Select the nominal web thickness = D/150 =1200/150=8mm

Increase the web thickness to 10 mm

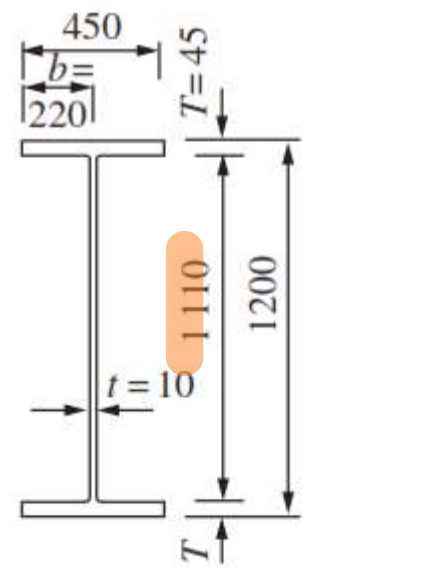


Figure 4 section selected

<u>*Stiffeners arrangement*</u>

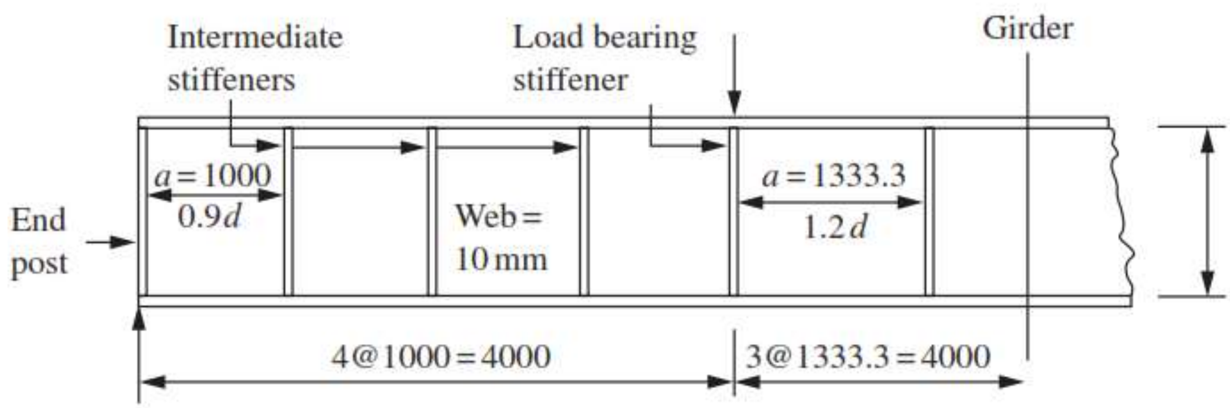


Figure 4 Stiffeners arrangement

Step 5 calculate section property and classification of section

Overall area of section Ag $= 2*(45x450) + 1110*10 = 516000\ mm^2$

Av area for shear $= \quad d*t = 1110*10 = \quad 11100\ mm^2$

$Ix = 10*1110^3/12+(450*45^3/12)*2+(450*45)*(1110/2+45/2)^2*2$

$= 7.9*10^9\ mm^4$

$rx = SQRT\ (7.9*10^9/516000) = 123\ mm$

- *classification for flange b/T= 220/45= 4.8*
  but Py = 255 N/mm3 since T =45 mm   table 9
  $\epsilon =$ 〚(275/255)〛 ^0.5

Outstand element of compression flange	Rolled section	b/T	9ε	10ε	15ε
	Welded section	b/T	8ε	9ε	13ε

From table 11 the flanges are classified as Plastic

- *Classification for web*
  d/t=1110/10=111

Web of an I-,	Neutral axis at mid-depth	d/t	80ε	100ε	120ε

From table 11 the web is classified as compact

Step 6 check minimum thickness of web

6.1 Minimum web thickness for serviceability During handling in site for erection

To avoid serviceability problems:

a) for webs without intermediate stiffeners: $t \geq d/250$;

b) for webs with transverse stiffeners only:

— where stiffener spacing $a > d$: $t \geq d/250$;

— where stiffener spacing $a \leq d$: $t \geq (d/250)(a/d)^{0.5}$;

Our case with stiffener spaced a =1000mm $< d = 1110\ mm$

$t \geq (d/250)(a/d)^{0.5}$; $=(1110/250)*(1000/1110)^{0.5}=4.2\ mm$

6.2 Minimum web thickness to avoid compression flange buckling

To avoid the compression flange buckling into the web:

a) for webs without intermediate stiffeners: $t \geq (d/250)(p_{yf}/345)$;

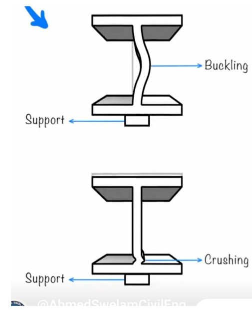
b) for webs with intermediate transverse stiffeners:

— where stiffener spacing $a > 1.5d$: $t \geq (d/250)(p_{yf}/345)$;

— where stiffener spacing $a \leq 1.5d$: $t \geq (d/250)(p_{yf}/455)^{0.5}$;

where

p_{yf} is the design strength of the compression flange.



Our case $p_{yf} = 255\ N/mm2$

— where stiffener spacing $a \leq 1.5d$: $t \geq (d/250)(p_{yf}/455)^{0.5}$;

$$t \geq {}^{1110}/_{250} \times \left(\frac{255}{455}\right)^{.5} = 3.32\ mm$$

t selected 10 mm *ok*

7.1 web not susceptible
7.2 web susceptible

Step 7 check if the Web susceptibility to shear buckling

<u>7.1 web is not susceptible to shear buckling if d/t ≤ 62∈</u>

∈=(275/py)

The moment capacity of the cross-section should be determined using BS 5950 clause 4.2.5.

7.1.1 Low shear

Provided that the shear force Fv does not exceed 60 % of the shear capacity Pv

Fv< 60% Pv

$$P_v = 0.6 p_y A_v$$

- *for class 1 plastic or class 2 compact cross-sections:*

OUR EXAMPLE

$$M_c = p_y S$$

- *for class 3 semi-compact sections:*

$$M_c = p_y Z$$

- *for class 4 slender cross-sections:* ***try to avoid***

$$M_c = p_y Z_{eff}$$

7.1.2 High shear

Where $F_v > 0.6P_v$:

- *for class 1 plastic or class 2 compact cross-sections:*

OUR EXAMPLE

$$M_c = p_y(S - \rho S_v)$$

Where S= section plastic modulus

- *for class 3 semi-compact cross-sections:*

$$M_c = p_y(Z - \rho S_v/1.5)$$

- *for class 4 slender cross-sections: :* ***try to avoid***

in which Sv is obtained from the following:

$$\boxed{Sv = td/4}$$

and ρ is given by:

$$\rho = [2(F_v/P_v) - 1]^2$$

$$S = 2\left(\frac{A_f}{2} \times \frac{h}{2}\right) + \frac{th^2}{4}$$

7.2 The web is susceptible to shear buckling if d/t > 62∈

The moment capacity of the cross-section should be determined taking account of the interaction of shear and moment *Reduced Moment capacity*

Our case

$\in = \left(\frac{275}{275}\right)^{0.5} = 1,$ *d/t= 111 >62* *d/t =1110/10 =111*

web is susceptible to shear buckling

7.2.1 Low shear if applied shear Fv < 0.6Vw

where Vw is the simple shear buckling resistance and given by

$$V_w = dtq_w$$

where

d	is the depth of the web;
q_w	is the shear buckling strength of the web;
t	is the web thickness.

The shear buckling strength qw should be obtained from Table 21 depending on the values of d/t and a/d where a is the stiffener spacing. For webs without intermediate stiffeners a/d should be taken as infinity.∞

Table 21 — Shear buckling strength q_w (N/mm²) of a web

1) Grade S 275 steel, thickness ≤ 16 mm — design strength p_y = 275 N/mm²															
d/t	Stiffener spacing ratio a/d														
	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	∞
55	165	165	165	165	165	165	165	165	165	165	165	165	165	165	165
60	165	165	165	165	165	165	165	165	165	165	165	165	165	165	165
65	165	165	165	165	165	165	165	165	165	165	165	165	165	165	161
70	165	165	165	165	165	165	165	165	165	165	164	162	160	158	155
75	165	165	165	165	165	165	165	165	163	160	158	156	154	152	148

Now in our solved example

Web depth/thickness ratio d/t = 111
Stiffener spacing/web depth ratio
a/d = 1000/1110 = 0.9
From Table 21 in the code the shear buckling strength:

qw= 143 N/mm2

simple Shear buckling resistance:

$Vw= 143 \times 10 \times 1110/10^3= 1587.3kN$

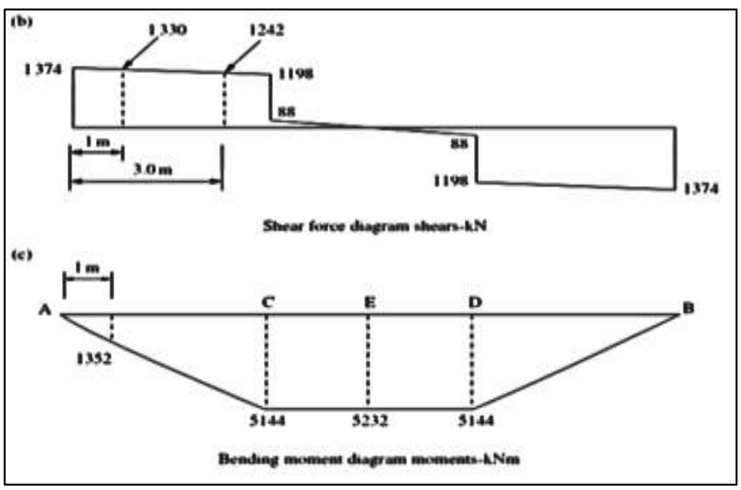
60% Vw= 952 kN

Refer to figure 3

I. *Fv at 1m = 1330 kN* *with Mu =1352 kN.m*
II. *Fv at 3m = 1242 kN* *with Mu =3858 kN.m*
III. *Fv at 4m = 1198 kN* *with Mu =5144kN.m*

Greater than *Vw= 952 kN*

High Shear

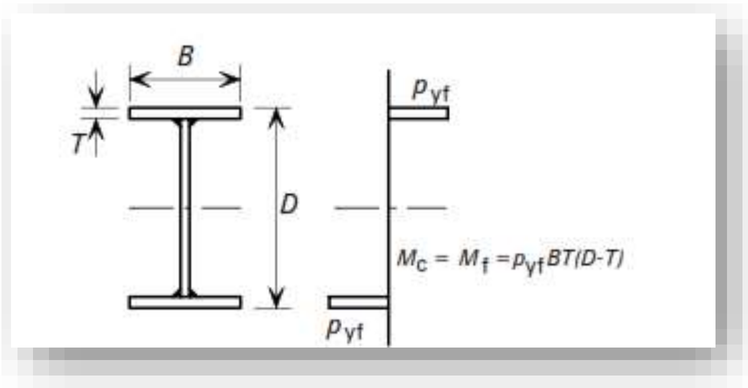


Shear force diagram shears-kN
Bending moment diagram moments-kNm

The moment capacity should be > Mu @III = 5144 kN.m in all cases Since

Fv>0 .6 Vw the section is considered with high shear in section I,II,III *high shear — "flanges only" method:*

If the applied shear is greater than 60% of Vw, the section can conservatively be designed by taking the entire shear on the web and the moment on the flanges. The uniform stress in the flanges should not exceed pyf, as shown in Figure 5. For this approach, the flanges must not be Class 4 slender



Moment capacity of plate girders with d/t ≥ 62ε and high shear using the "flanges only" method

Now in our solved example

Mc =Mf(flang moment capacity) =p_{yf} *(BT(D-T)) =255x(450x40)(1200+40)/*10^6

= 5324 kN.m >Mu =5144 kN.m

THE PLATE GIRDER MOMENT CAPACITY IS WELL SATISFIED

١ مقدمة

غالبًا ما تتجاوزالعزوم وقوى القص المرتبطة بحمل أحمال كبيرة على بحورواسعه سعةالقطاعات المنتجه من المصانع. في هذه الحالة، يمكن تصنيع قطاعات صفائحية، مع تصميم أبعادها لتوفير نسبة عالية من القوة إلى الوزن.

في القطاعات المُصنّعة

1.الوظيفة الأساسية للصفيحه العلويه او السفليه هي مقاومة قوى الضغط والشد المحورية

الناتجة عن عزم الانحناء. Top flange+bottom flange

.
الوظيفة الأساسية للعصب web هي مقاومة قوة القص.

لتصميم قطاعات صفائحية فعّالة، يجب زيادة عمق العصب d قدر الإمكان لتوفير أقل قوةفي الصفائح العلويه ةالسفليه عند عزم انحناء مُحدد. لتقليل الوزن الذاتي، يجب تقليل سمك النسيج t إلى الحد الأدنى.

نتيجةً لهذه المتطلبات، يتمتع النسيج بنسبة d/t عالية، ويميل إلى الانبعاج عند القص في حال عدم توفير مُقوّ يات. **انظر الشكل**

للحصول على تصميم اقتصادي، ينبغي الاستفادة من احتياطي القوة بعد الانبعاج، المعروف باسم "تأثير مجال الشد"tension field action.
يسمح الكود BS 5950-1 بأخذ هذا الاحتياطي في الاعتبار.
من المحتم أن تؤدي زيادة كفاءة التصاميم وفقًا للمعيار BS 5950-1 إلى تعقيد إضافي في حسابات التصميم. هناك متطلبات خاصة لأطراف العوارض الصفيحية لتثبيت "تأثير مجال الشد". يظهر في الشكل 1 عارضة صفيحية نموذجية.