

TRAMPOLINS - EMÍLIO BAUMGART**1931 - 2020 TRAMPOLIM DO FLUMINENSE F.C. / RJ - PROJETO 142 / 359 DE EMÍLIO BAUMGART****FOTOS EM 2018**

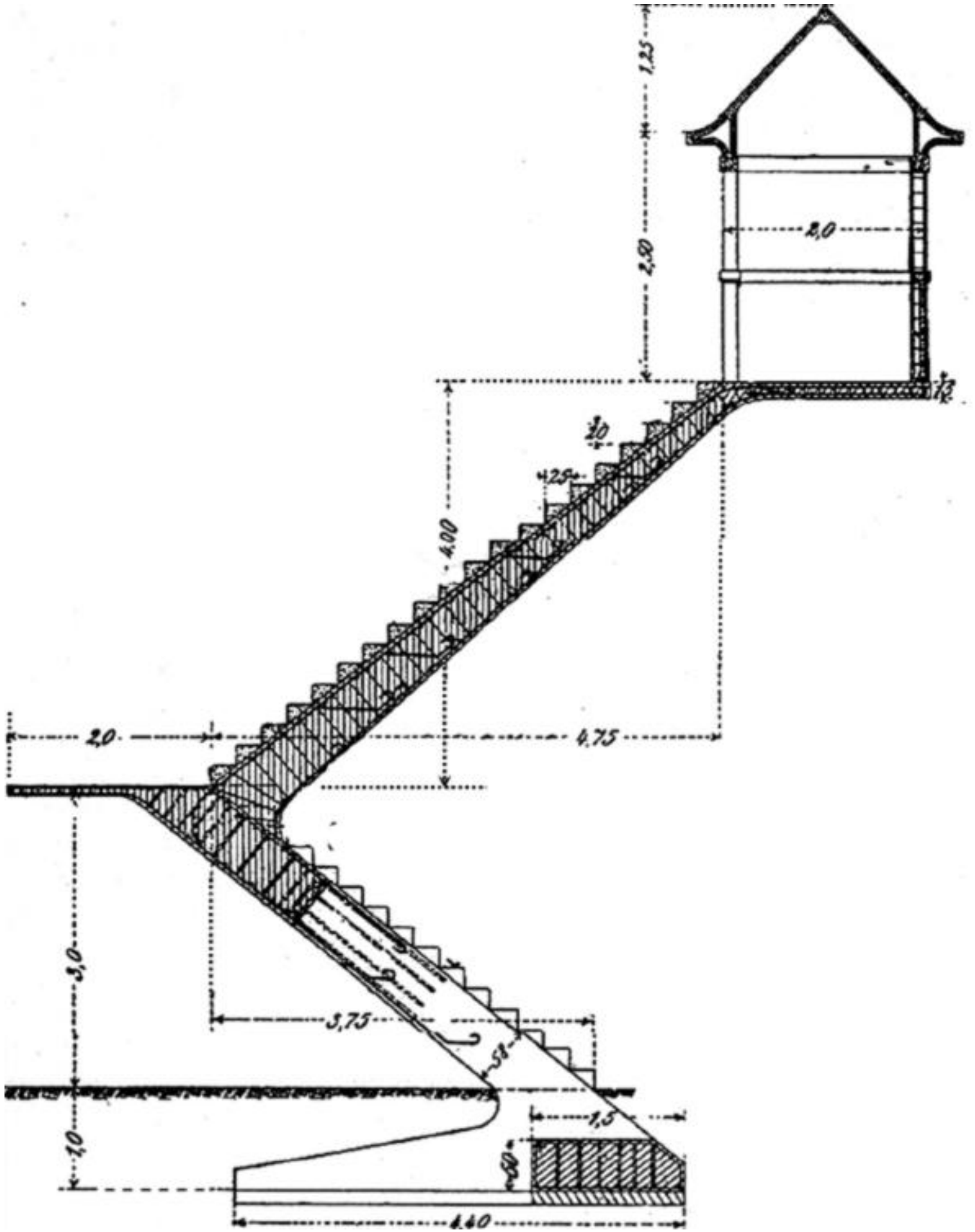
**1931 - 2020 TRAMPOLIM DO FLUMINENSE F.C. / RJ - PROJETO 142 / 359 DE EMÍLIO BAUMGART
VIGAS TRANSVERSAIS EM BALANÇO COM TORÇÃO.**



No livro de **Emil Moersch**, “**Teoría y Práctica del Hormigón Armado**” , volume III, tradução do original alemão, é mostrada uma escada executada segundo projeto do Prof. Hohnke da Universidade de Danzig.

Similar a essa escada Baumgart, em um dos seus trampolins, deixou apenas os degraus como estrutura.





1934 TRAMPOLIM de CONCRETO ARMADO - GYMNASIO VERA CRUZ

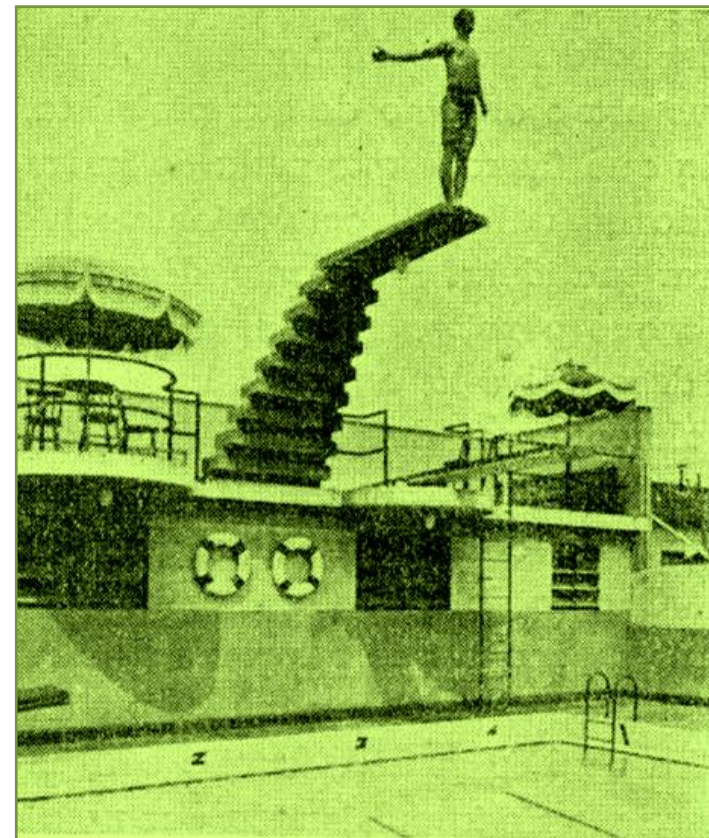
na rua São Francisco Xavier Nº 417 no Maracanã / RJ

PROJETO 193 / 359 DE EMÍLIO BAUMGART

A NOITE - 17/3/1936 e DIÁRIO DE NOTÍCIAS -27/3/1936

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Palestra do Prof. Arthur Eugênio Jermann - 1944



- *"Quando salta um atleta a prancha de concreto armado vibra como uma tábua"*

"A grande flexibilidade desse trampolim, conseguida com detalhes, que indicam um profundo conhecimento do material, torna desnecessária a geralmente imprescindível tábua de madeira, que se vê ao lado."

Mais tarde o trampolim foi demolido para a construção de um prédio.

PROF. ARTHUR EUGENIO JERMANN

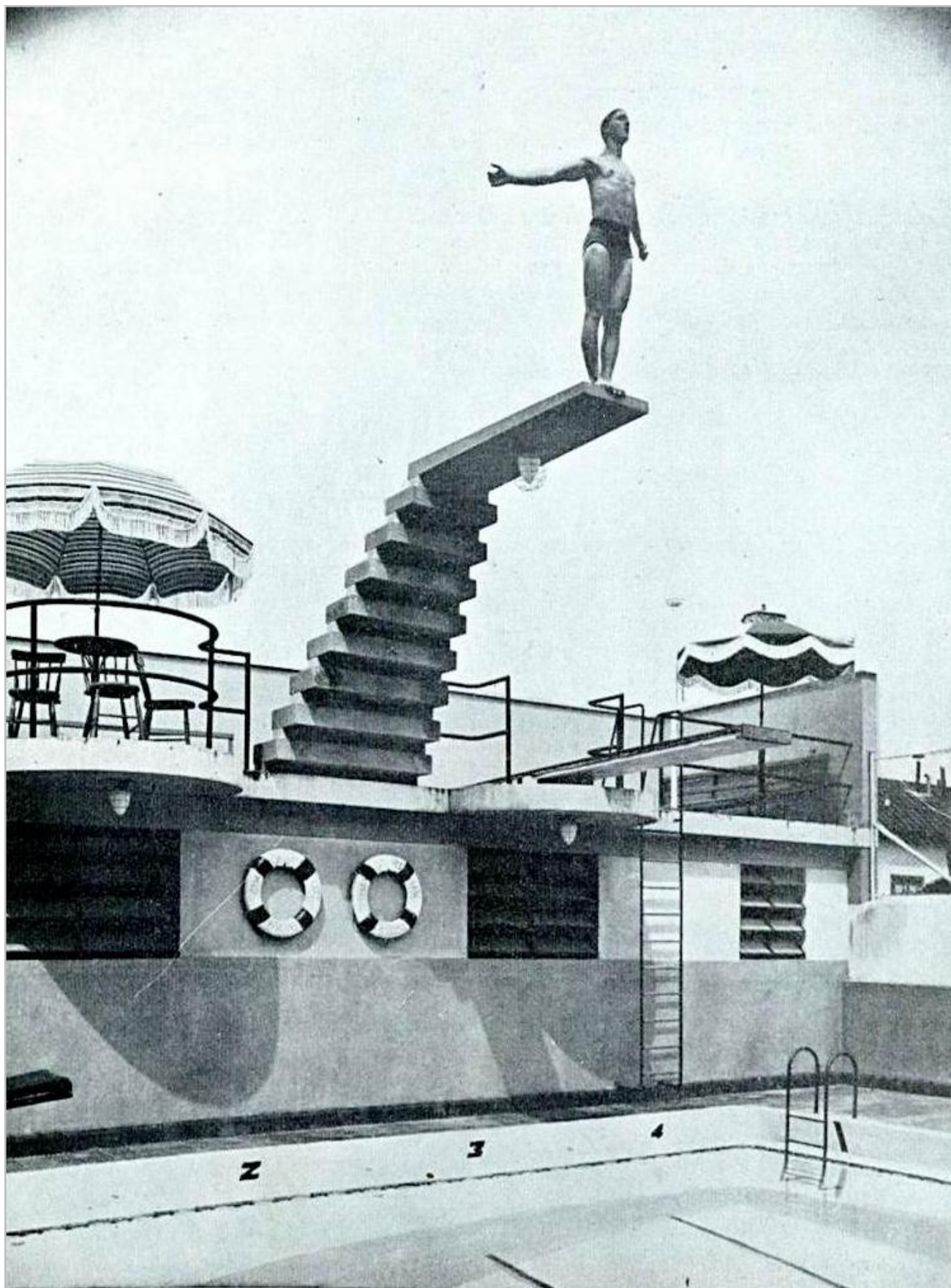


Fig. 10 — Trampolim de concreto armado, na piscina do Ginásio Vera Cruz, no Rio de Janeiro

PÁGINA EM BRANCO

1936

TRAMPOLIM EM CACHOEIRO DE ITAPEMIRIM / ES

PROJETO 225 / 359 DE EMÍLIO BAUMGART

SIMILAR AO TRAMPOLIM DO GYMNASIO VERA CRUZ / RJ

*cópias dos desenhos cedidas por
Eng. Eduardo Carvalho Guimarães*

Um trampolim de Emílio Baumgart em Cachoeiro de Itapemirim / ES, feito em 1936.

Cópias dos desenhos de fôrma e de armadura cedidas pelo Eng. Eduardo Carvalho Guimarães, a quem agradecemos.

Notável a armadura dos degraus com ferros de 5mm em "**espiraes**" com passo de 1,25cm e com ferros de 6,3 mm em "**espiraes**" com passo de 2,5cm

Todos os degraus são diferentes uns dos outros, com as dimensões diminuindo no sentido da ponta do balanço.

Trabalho grande para fazer as fôrmas e as armaduras.

Cobrimentos de 1,0 cm

AÇO DOCE COMUM TENSÃO DE RUPTURA : 3700 kgf.cm²

LIMITE DE ELASTICIDADE : 2400 kgf/cm² ;

Tensão admissível em serviço =1200 kgf/cm²

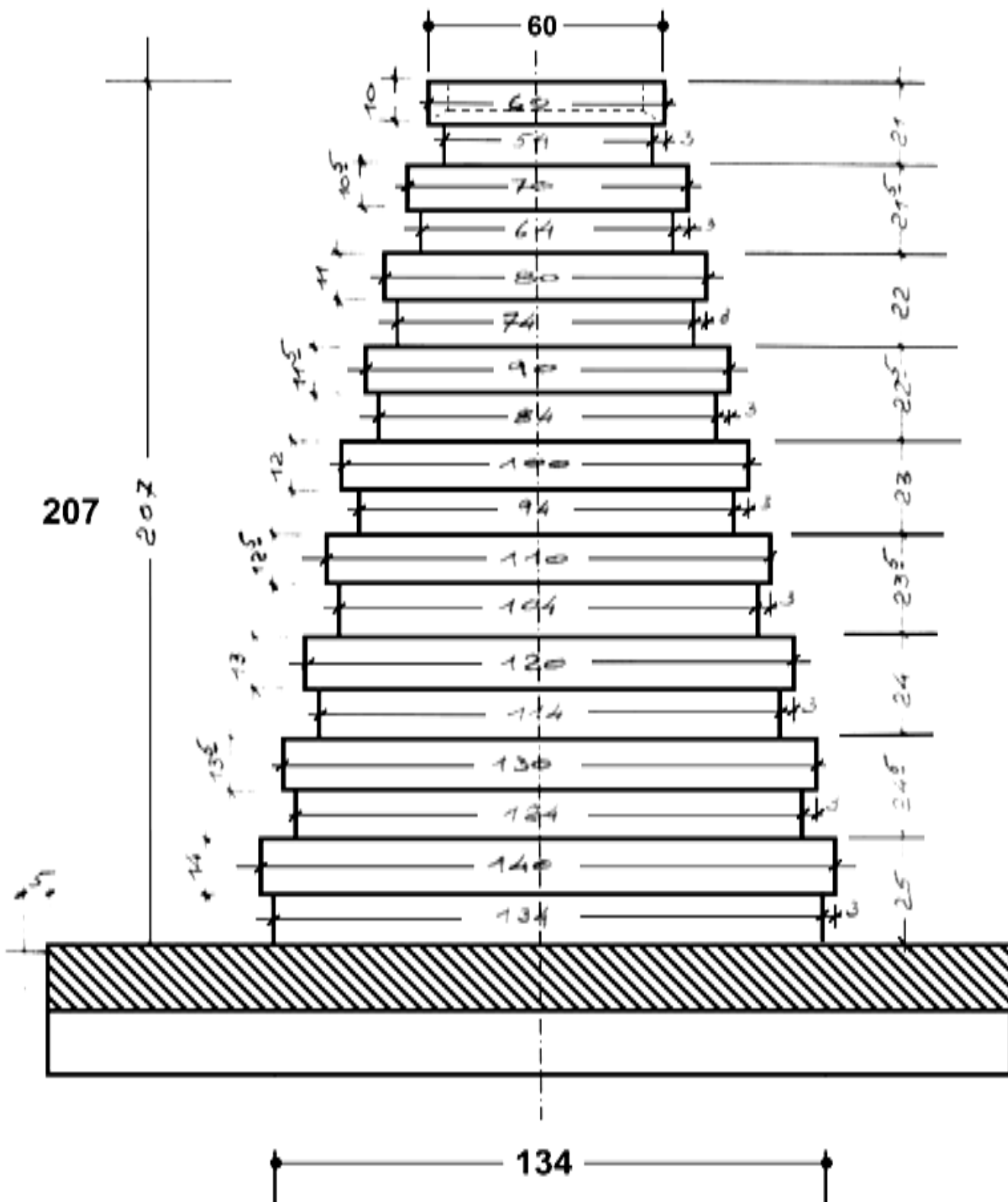
Calculei a armadura necessária. Resultou igual à armadura existente no projeto.

Eduardo Thomaz

1936 - TRAMPOLIM EM CACHOEIRO DE ITAPEMIRIM / ES

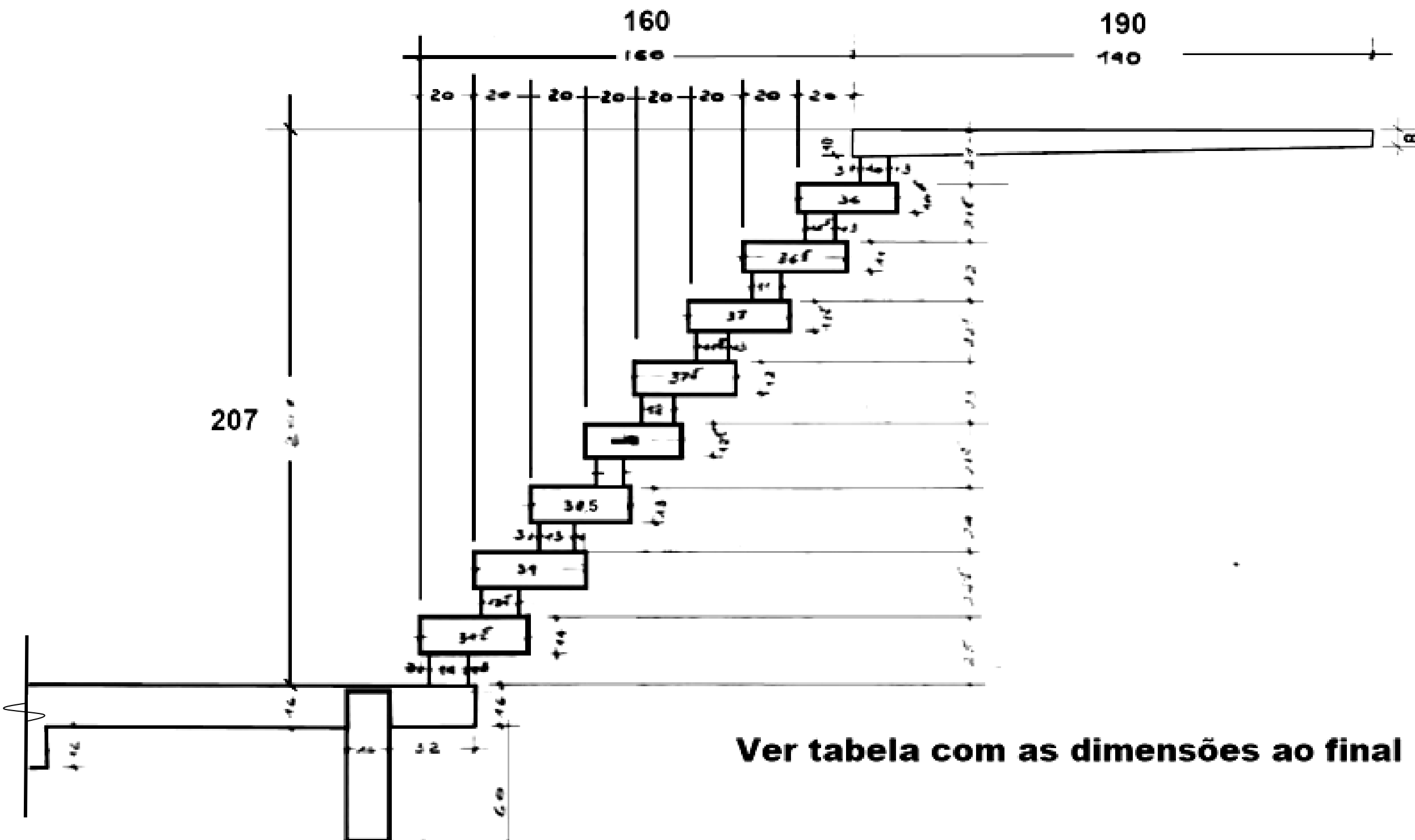
M. F. Souza Eng. Civil	16-11-36	1:20 formas do trampolim
consultorio tecnico Emilio H. Baumgart	689-01	Cachoeira do Itapemirim

VISTA FRONTAL



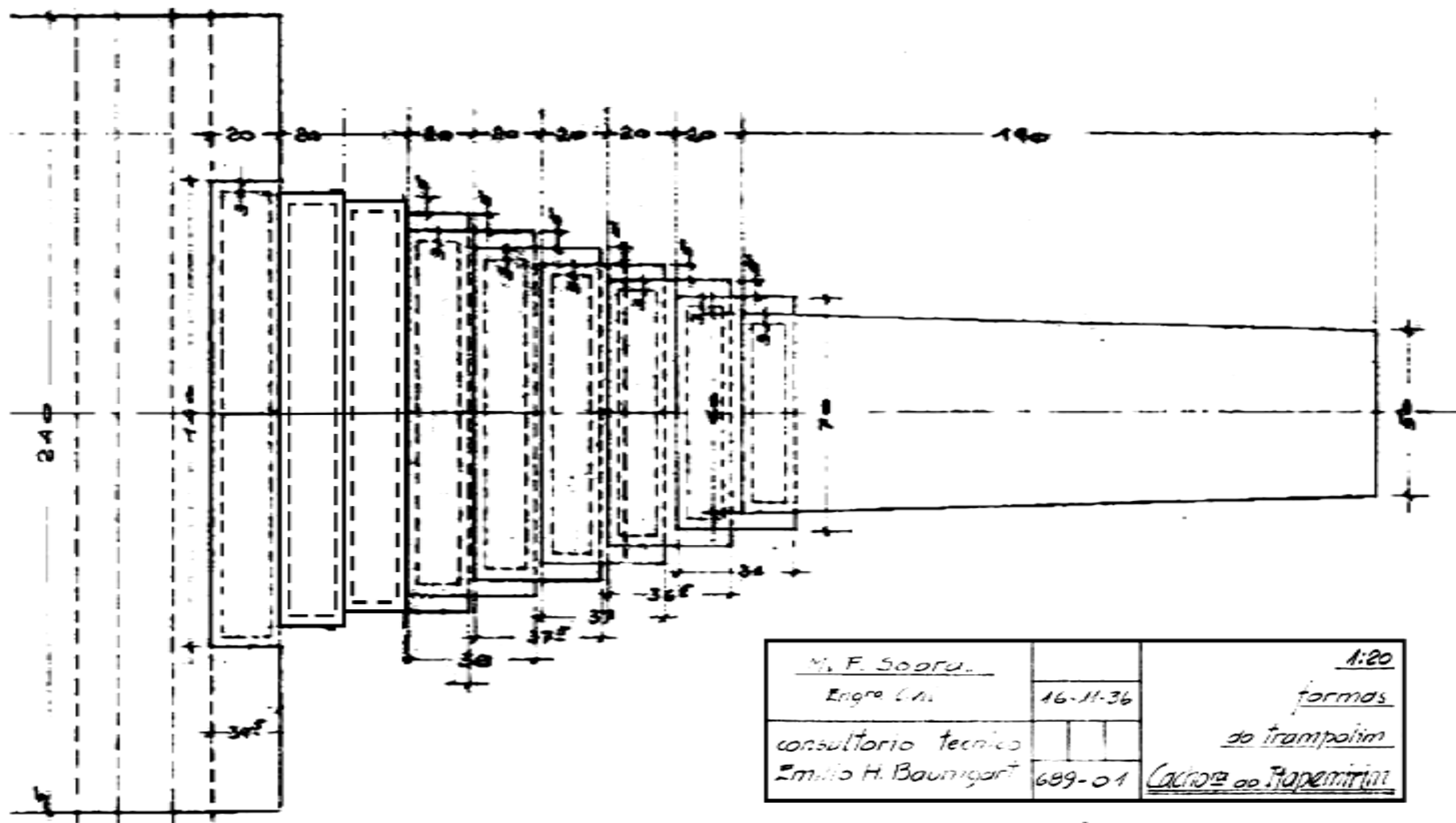
Ver tabela com as dimensões ao final

VISTA LATERAL



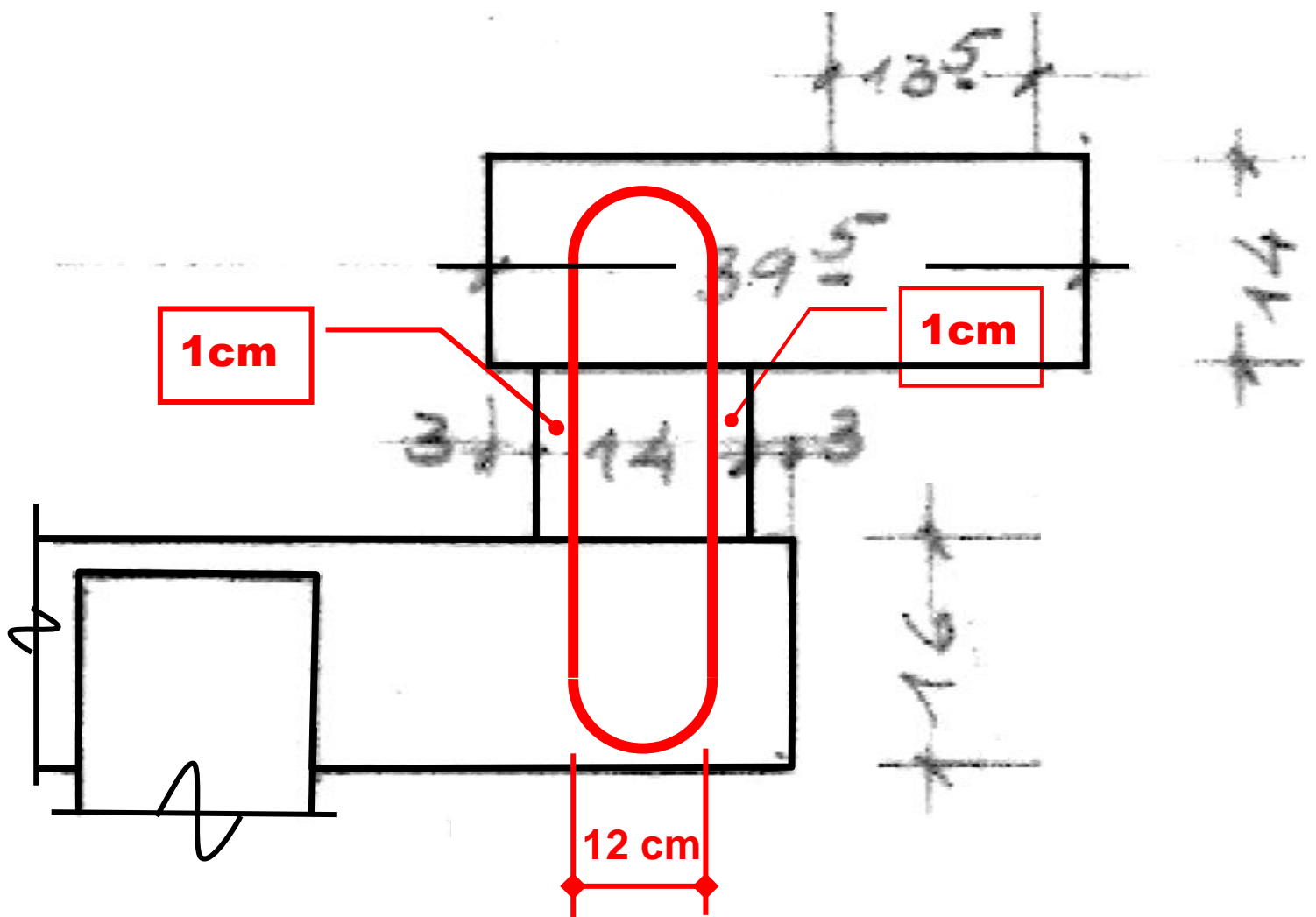
Ver tabela com as dimensões ao final

PLANTA



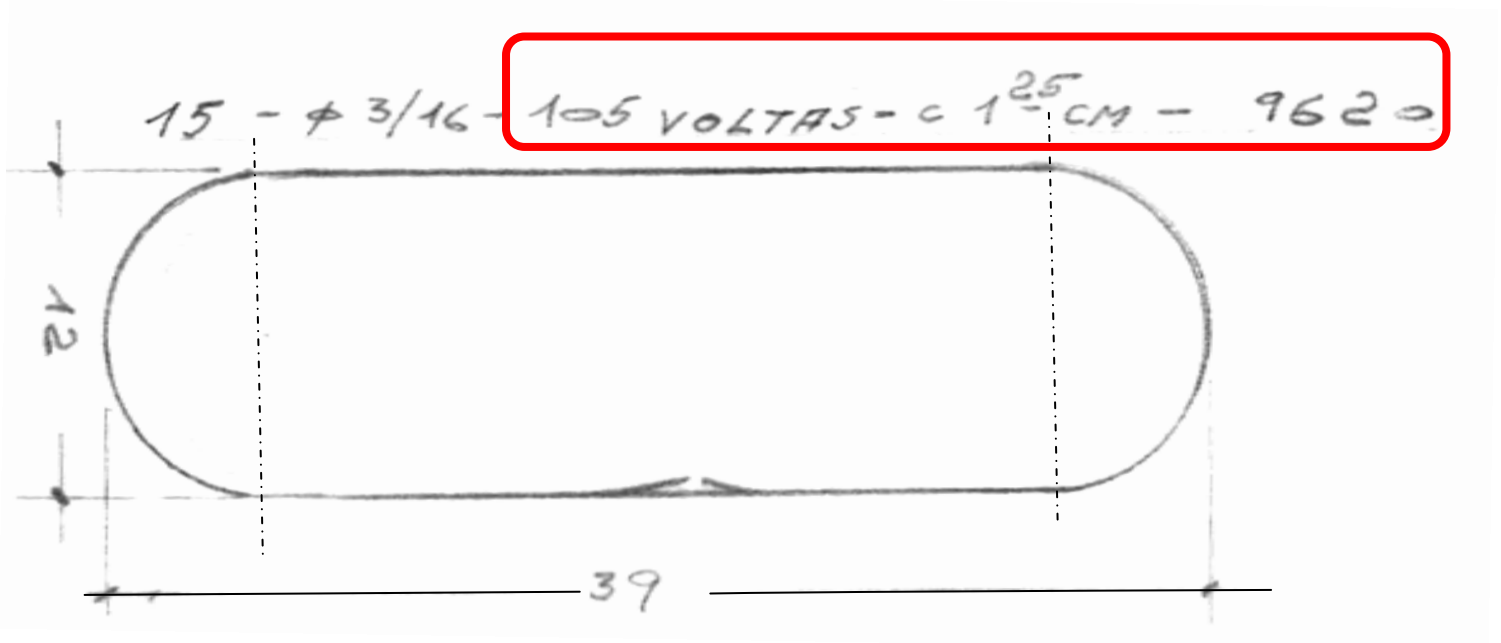
<u>M. F. Sobral</u> Engro. Civil	20-M-36	2ª Parte	1:5
consultorio tecnico Emilio H. Baumgart	689-03		armação do trampolim
		Cachoeira do Itapemirim	

ARMADURA "EM ESPIRAES" NO PRIMEIRO DEGRAU



ARMADURA "EM ESPIRAES" NO 1º DEGRAU

(armadura girada 90 graus para facilitar a leitura)



$$\text{UMA VOLTA} = 2 \times 27\text{cm} + \text{Pi} \times 12 = 54 + 37,7 = 91,7 \text{ cm}$$

$$105 \text{ espiras} = 105 \times 91,7 \text{ cm} = 9628 \text{ cm} = 96 \text{ m} \quad \text{OK}$$

$$105 \text{ espiras a cada } 1,25\text{cm} = 104 \times 1,25\text{cm} = 130\text{cm}$$

A largura do degrau vale 134 cm

$$\text{cobrimento lateral} = (134 \text{ cm} - 130\text{cm}) / 2 = 2\text{cm}$$

$$\text{Espaço livre entre ferros} = 1,25\text{cm} - (3/16" = 5\text{mm}) = 7,5\text{mm}$$

$$\text{Área de ferro tracionado} = 105 \times 0,2\text{cm}^2 = 21\text{cm}^2$$

AÇO DOCE COMUM TENSÃO DE RUPTURA : 3700 kgf/cm²

LIMITE DE ELASTICIDADE : 2400 kgf/cm² ;

$$\underline{\sigma_{adm} \text{ em serviço} = 1200 \text{ kgf/cm}^2}$$

$$\text{Força de tração no aço em serviço} = 21 \text{ cm}^2 \times (\sigma_{adm} = 1200 \text{ kgf/cm}^2) = 25,2 \text{ ton}$$

Com a resultante de compressão no concreto atuando muito próximo da armadura comprimida (ver cálculo adiante) temos $z = 12\text{cm}$:

$$\text{Momento fletor admissível em serviço} = 25,2 \text{ ton} \times 0,12\text{m} = \mathbf{3,14 \text{ t.m}}$$

MOMENTO FLETOR ATUANTE EM SERVIÇO

$$\text{PESO PRÓPRIO DA ESTRUTURA} = 1,58 \text{ t.m}$$

$$+ \text{ UM SALTADOR NA PONTA DO BALANÇO} = 0,34 \text{ t.m}$$

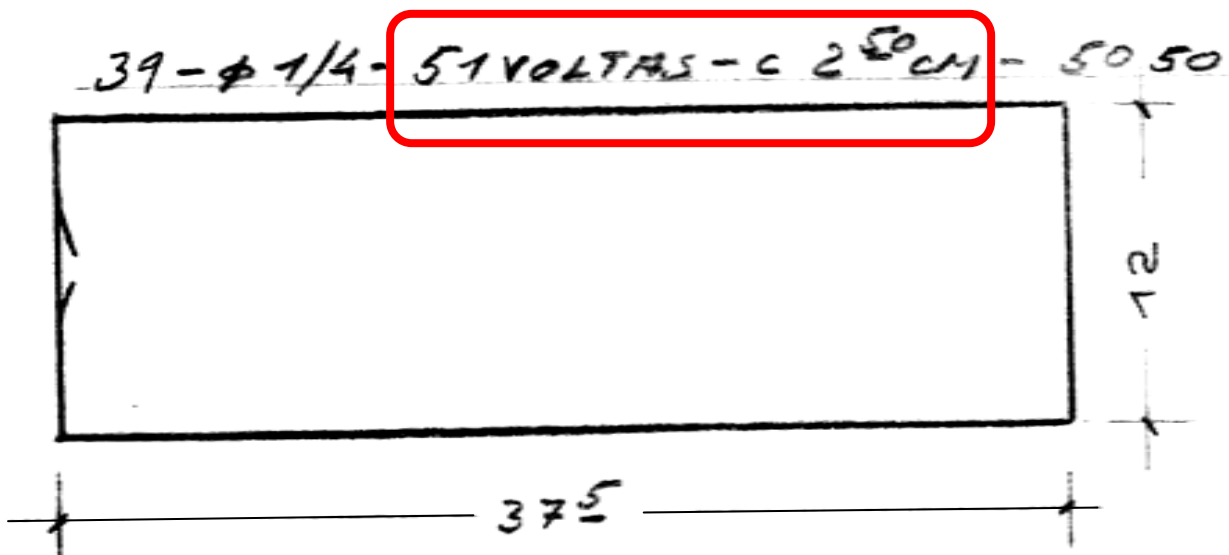
ou

$$+ 4 \text{ PESSOAS NO BALANÇO} = 0,75 \text{ t.m}$$

$$\text{MOMENTO FLETOR MÁXIMO EM SERVIÇO} = 1,58 + 0,75 = \mathbf{2,33 \text{ t.m}}$$

LEVANDO EM CONTA A FORÇA NORMAL NA FLEXÃO COMPOSTA, COM COMPRESSÃO, OBSERVA-SE QUE O FATOR DIMENSIONANTE NESTA SEÇÃO FOI A TENSÃO LIMITE NO CONCRETO E NÃO A TENSÃO NO AÇO TRACIONADO (VER CÁLCULO AO FINAL)

ARMADURA DO PRIMEIRO PISO



$$\text{UMA VOLTA} = 2 \times 37,5\text{cm} + 2 \times 12 = 75 + 24 = 99 \text{ cm}$$

$$51 \text{ VOLTAS} = 51 \times 99 \text{ cm} = 5050 \text{ cm} = 51\text{m} \quad \text{OK}$$

$$51 \text{ voltas a cada } 2,5\text{cm} = 51 \times 2,5\text{cm} = 128\text{cm}$$

A largura do degrau vale 140 cm

$$\text{cobrimento} = (140 \text{ cm} - 128\text{cm}) / 2 = 6\text{cm}$$

$$\text{Espaço livre entre ferros} = 2,5\text{cm} - (1/4" = 6,3\text{mm}) = 19\text{mm}$$

$$\text{Área de ferro tracionado} = 51 \times 0,32\text{cm}^2 = 16\text{cm}^2$$

AÇO DOCE COMUM TENSÃO DE RUPTURA : 3700 kgf/cm²

LIMITE DE ELASTICIDADE : 2400 kgf/cm² ;

$$\underline{\sigma_{adm} \text{ em serviço} = 1200 \text{ kgf/cm}^2}$$

$$\text{Força de tração no aço em serviço} = 16 \text{ cm}^2 \times (\sigma_{adm} = 1200 \text{ kgf/cm}^2) = 19,2 \text{ ton}$$

$$\text{Momento fletor admissível em serviço} = 19,2 \text{ ton} \times 0,12\text{m} = \mathbf{2,3 \text{ t.m}}$$

MOMENTO FLETOR ATUANTE EM SERVIÇO

$$\text{PESO PRÓPRIO DA ESTRUTURA} = 1,58 \text{ t.m}$$

$$+ \text{ UM SALTADOR NA PONTA DO BALANÇO} = 0,34 \text{ t.m}$$

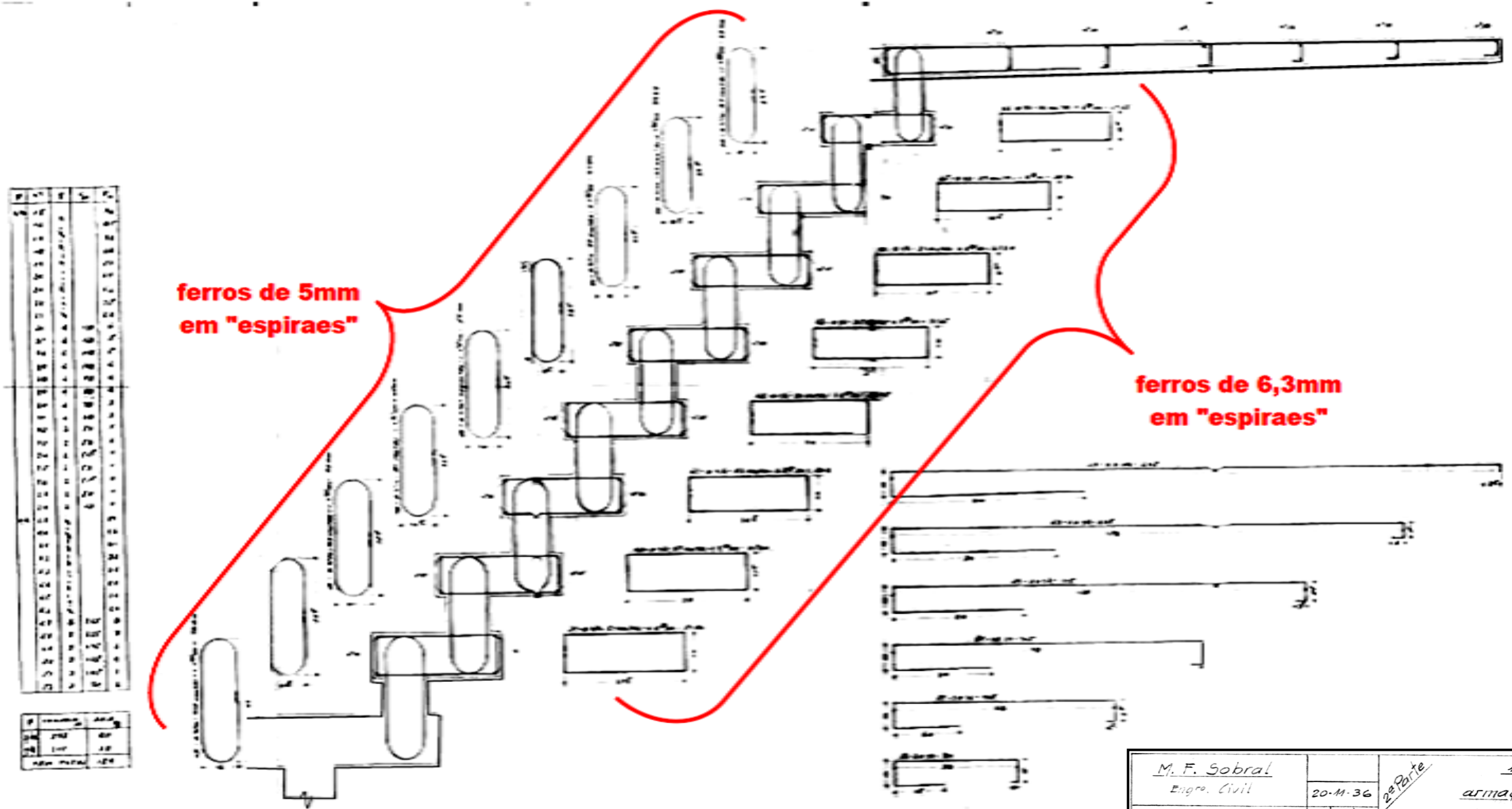
ou

$$+ 4 \text{ PESSOAS NO BALANÇO} = 0,75 \text{ t.m}$$

$$\text{MOMENTO FLETOR MÁXIMO EM SERVIÇO} = 1,58 + 0,75 = \mathbf{2,33 \text{ t.m} \quad \text{OK}}$$

+++

ARMADURA DA ESCADA COM FERROS $\phi 5\text{mm}$ e $\phi 6,3\text{mm}$



1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70
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M. F. Sobral		1:5
Engro. Civil	20.11.36	2ª Parte
consultorio tecnico		armação
Emilio H. Baumgart	689-03	do trampolim
		Cachoeira do Itapemirim

LISTA DE FERROS

ϕ	n°	q	C_{an}	t_M
3/16	15	E-S-P-1-R-A-E-S		96
	16		85	
17	78			
18	66			
19	57			
20	49			
21	42			
22	35			
23	28			
24	4		138	6
25	4		128	5
26	4		118	5
27	4		108	4
28	4		98	4
29	4		88	4
30	4		78	3
31	4		68	3
32	2		58	1
33	2		56	1
34	2		54 ⁵	1
35	2	53	1	
36	2	51 ⁵	1	

	36	2	51 ⁵	1
	37	2	50	1
	38	2	48	1
1/4	39	E-S-P-1-R-A-E-S		51
	40		46	
41	41			
42	36			
43	31			
44	28			
45	28			
46	20			
47	6		265	8
48	3		225	7
49	3		195	6
50	3		145	4
51	3		105	3
52	3		70	2

ϕ	COMPRIM. M	PESO Kg
3/16	577	81
1/4	311	78
PESO TOTAL		159

Como o volume de concreto é de 0,636 m³, a taxa de armadura é de 159 kg / 0,636m³ = 250 kg/m³. Muito alta.

Como o espaço livre entre as barras é muito pequeno o concreto não deve ter sido feito com pedras britadas. Na realidade era um argamassa com 350 kg de cimento por m³ de argamassa.

PÁGINA EM BRANCO

**VERIFICAÇÃO DE TENSÕES
NO CONCRETO E NA ARMADURA
SEGUNDO MÖRSCH**

1936 TRAMPOLIM DE EMÍLIO BAUMGART
EM CACHOEIRO DE ITAPEMIRIM
VERIFICAÇÃO DAS TENSÕES NO CONCRETO E NO AÇO
USANDO O ESTÁDIO II E AS FORMULAÇÕES DE MÖRSCH

CONCRETE-STEEL CONSTRUCTION

(DER EISENBETONBAU)

BY

PROFESSOR EMIL MÖRSCH

Of the Zurich Polytechnic, Zurich, Switzerland

AUTHORIZED TRANSLATION FROM THE

THIRD (1908) GERMAN EDITION, REVISED AND ENLARGED

BY

E. P. GOODRICH

Consulting Engineer

NEW YORK

THE ENGINEERING NEWS PUBLISHING COMPANY

LONDON: ARCHIBALD CONSTABLE AND COMPANY, LTD.

1909

The moment M , with reference to the centroid of the modified section, produces on the one side compressive stresses and on the other side tensile ones. In this case, however, the tensile stresses, since they represent only a decrease in the uniformly distributed compressive stress, are to be calculated as for a homogeneous section, in which the area of reinforcement is to be replaced by a concrete one $\frac{F_s}{E_b}$ times larger. It is thus necessary to calculate the moment of inertia J in the formula

$$\sigma = \frac{vM}{J},$$

from the expression

$$J = \frac{b}{3}u^3 + \frac{b}{3}(d-u)^3 + n F_s(h-u)^2 + n F_s'(u-h')^2.$$

Bending with axial compression is the usual stress condition in the sections of arches. In them the reinforcement is usually symmetrically arranged, so that the centroid of the whole section coincides with the axis of the arch and the calculation assumes a fairly simple form. The area of the modified section is then

$$F = bd + 2 n F_s,$$

and the moment of inertia is

$$J = \frac{b}{12}d^3 + 2 n F_s \left(\frac{d}{2} - c \right)^2.$$

If values are assumed for F and J , the same conditions exist in the reinforced section as regards the rib, as for a homogeneous section.

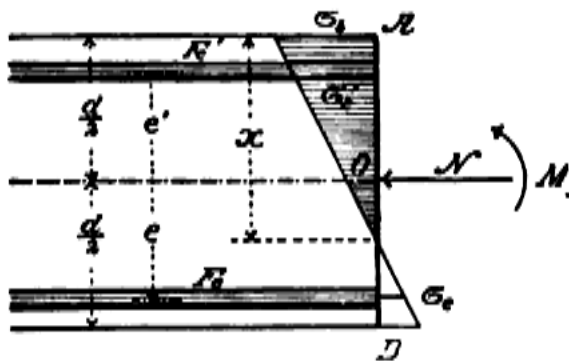


FIG. 107.

2. The resultant is supposed to have such an eccentricity that tensile stresses exist on one side of the section.

If these tensile stresses are insignificant, the calculation may be made exactly as in 1. If, however, they are appreciable, a special modulus of elasticity for tension must be introduced into the calculations. Usually, in order to obtain a proper safety factor, the tensile strength of the concrete is disregarded, as in simple flexure.

In Fig. 107, O represents the centroid of the concrete section to which the moment M is referred, and x is the distance of the neutral axis from the compression edge of the section. Then

$$N = \frac{\sigma_b}{2} b x + F_s' \sigma_c' - F_s \sigma_c, \quad \dots \dots \dots (1)$$

$$M = \frac{\sigma_b b x}{2} \left(\frac{d}{2} - \frac{x}{3} \right) + F_s' \sigma_c' c' + F_s \sigma_c e. \quad \dots \dots \dots (2)$$

BENDING WITH AXIAL FORCES

Further, because of the conservation of plane sections,

$$\sigma_e = \frac{E_e}{E_b} \sigma_b \frac{e + \frac{d}{2} - x}{x} = n \sigma_b \frac{e + \frac{d}{2} - x}{x}, \dots \dots \dots (3)$$

$$\sigma_{e'} = \frac{E_e}{E_b} \sigma_b \frac{e' - \frac{d}{2} + x}{x} = n \sigma_b \frac{e' - \frac{d}{2} + x}{x} \dots \dots \dots (4)$$

These four equations suffice for the determination of the four unknowns, x , σ_b , σ_e , $\sigma_{e'}$. If the external forces and given dimensions are used to calculate x the following equation of the third degree results, which can best be solved by trial.

$$\begin{aligned} &x^3 \frac{N}{6} - x^2 \left(\frac{Nd}{4} - \frac{M}{2} \right) + \frac{xn}{b} [M(F_{e'} + F_e) - N(F_{e'}e' - F_e e)] \\ &+ \frac{Mn}{b} \left[F_{e'} \left(e' - \frac{d}{2} \right) - F_e \left(e + \frac{d}{2} \right) \right] \\ &- \frac{Nn}{b} \left[F_{e'} e' \left(e' - \frac{d}{2} \right) + F_e e \left(e + \frac{d}{2} \right) \right] = 0. \end{aligned}$$

Then

$$\sigma_b = \frac{Nx}{\frac{bx^2}{2} + nF_{e'} \left(e' - \frac{d}{2} + x \right) - nF_e \left(e + \frac{d}{2} - x \right)}$$

As a rule, in arches and columns, the reinforcement is symmetrically arranged, and there are obtained, from equations (1) to (4), with $F_e = F_{e'}$ and $e' = e$, the following relations:

$$N = \sigma_b \frac{bx}{2} + F_e (\sigma_{e'} - \sigma_e), \dots \dots \dots (5)$$

$$M = \sigma_b \frac{bx}{2} \left(\frac{d}{2} - \frac{x}{3} \right) + e F_e (\sigma_{e'} + \sigma_e), \dots \dots \dots (6)$$

$$\sigma_e = n \sigma_b \frac{e + \frac{d}{2} - x}{x}, \dots \dots \dots (7)$$

$$\sigma_{e'} = n \sigma_b \frac{e - \frac{d}{2} + x}{x}, \dots \dots \dots (8)$$

while the equation for the solution of x takes the form

$$x^3 \frac{N}{6} - x^2 \left(\frac{Nd}{4} - \frac{M}{2} \right) + 2x M n \frac{F_e}{b} - n \frac{F_e}{b} (M d + 2N e^2) = 0,$$

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CONCRETE-STEEL CONSTRUCTION

OR

$$x^3 - 3x^2 \left(\frac{d}{2} - \frac{M}{N} \right) + 12x \frac{M}{N} n \frac{F_e}{b} - 6 \frac{n F_e}{b} \left(\frac{M}{N} d + 2e^2 \right) = 0.$$

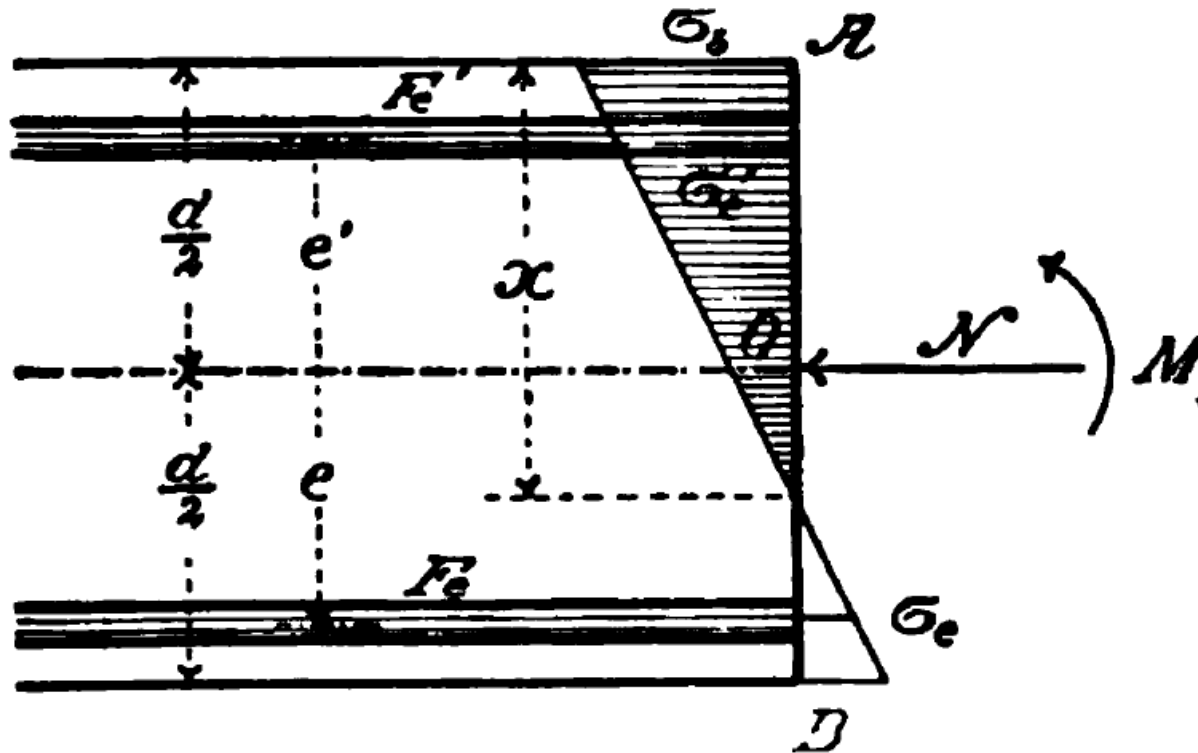


FIG. 107.

When x is ascertained, the stresses may be found by inserting the value of x in equations (8), (7), (6), and (5), and

$$\sigma_b = \frac{N}{\frac{bx}{2} + \frac{nF_e}{x}(2x - d)}$$

$$\sigma_e = n \sigma_b \frac{e + \frac{d}{2} - x}{x},$$

$$\sigma_e' = n \sigma_b \frac{e - \frac{d}{2} + x}{x}.$$

DADOS DO TRAMPOLIM ITAPEMIRIM

PRIMEIRO DEGRAU : $d=14\text{cm}$; $b=134\text{ CM}$

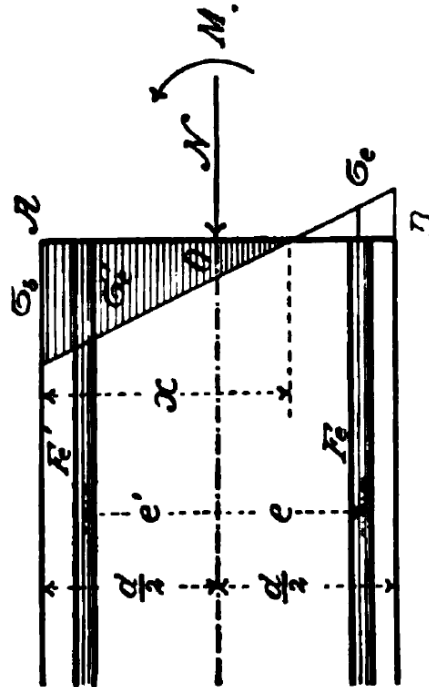


FIG. 107.

VER PLANILHA COM A GEOMETRIA DOS DEGRAUS ao final

$$N = 1,89 \text{ ton}$$

$$M = 2,33 \text{ ton.m}$$

$$M/N = 2,33 \text{ tm} / 1,89 \text{ t} = 1,233 \text{ m} = 123,3 \text{ cm}$$

$$e = e' = 6 \text{ cm}$$

$$d = 14 \text{ cm}$$

$$b = 134 \text{ cm}$$

$$F_e = F_e' = 105 \text{ ferros } 5 \text{ mm} = 105 \times 0,2 \text{ cm}^2 = 21 \text{ cm}^2$$

$$n = E_{\text{aço}} / E_{\text{concreto}} =$$

$$= 2\,100\,000 \text{ kgf/cm}^2 / 200\,000 \text{ kgf/cm}^2 = 10$$

APLICANDO

$$x^3 - 3x^2 \left(\frac{d}{2} - \frac{M}{N} \right) + 12x \frac{M}{N} n \frac{F_e}{b} - 6 \frac{nF_e}{b} \left(\frac{M}{N}d + 2e^2 \right) = 0.$$

$$-3x^2 \left(\frac{d}{2} - \frac{M}{N} \right)$$

$$- 3 \times (d/2 - M/N) = - 3 \times (14\text{cm}/2 - 123,3\text{cm} = -116,3\text{cm}) = \underline{\underline{+ 348,9\text{cm}}}$$

$$+ 12x \frac{M}{N} n \frac{F_e}{b}$$

$$12 \times (M/N) \times n \times (F_e/b) =$$

$$+12 \times 123,3\text{cm} \times 10 \times (21\text{cm}^2 / 134\text{cm}) = \underline{\underline{+ 2318,8\text{cm}^2}}$$

$$-6 \frac{nF_e}{b} \left(\frac{M}{N}d + 2e^2 \right)$$

$$- 6 \times [(10 \times 21\text{cm}^2)/134\text{cm}] \times (123,3\text{cm} \times 14\text{cm} + 2 \times 6\text{cm} \times 6\text{cm}) =$$

$$- 9,403 \times (1726,2 + 72) = \underline{\underline{- 16908,5}}$$

$$x^3 + 348,9x^2 + 2318,8x - 16908,5 = 0$$

RESOLVENDO COM O EXCEL

$$\underline{\underline{X = 4,37534 \text{ cm}}}$$

Tensão no concreto

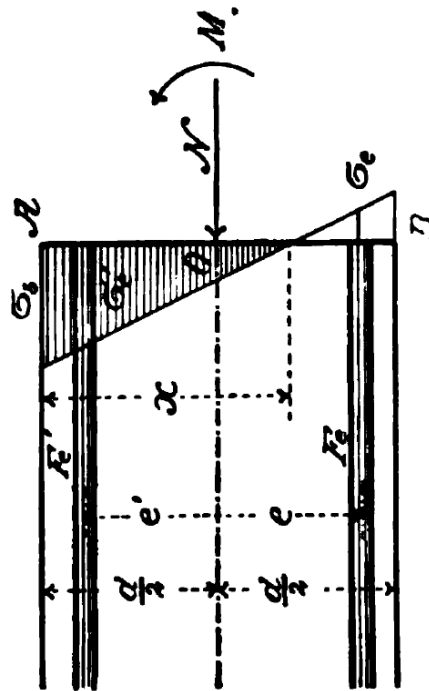


FIG. 107.

$$\sigma_b = \frac{N}{\frac{bx}{2} + \frac{nF_c}{x}(2x - d)}$$

$$N = 1,89 \text{ ton}$$

$$M = 2,33 \text{ ton.m}$$

$$x = 4,37534$$

$$\text{sigma concreto} = \frac{1890 \text{ kgf}}{\frac{134 \text{ cm} \times 4,37534 \text{ cm}}{2} + \frac{10 \times 21 \text{ cm}^2}{4,37534 \text{ cm}} \times (2 \times 4,37534 - 14)} =$$

$$= \frac{1890 \text{ kgf}}{293,148 \text{ cm}^2 + 47,996 \text{ cm} \times (-5,2493 \text{ cm}^2)} =$$

$$= \frac{1890 \text{ kgf}}{41,2025972 \text{ cm}^2} = 45,871 \text{ kgf/cm}^2 = \text{CONCRETO A.350 ver adiante}$$

OBSERVAÇÃO :

" REGULAMENTO PARA CONSTRUÇÕES EM CONCRETO ARMADO", RJ /1931.

DOSAGEM DO CONCRETO § 35

B - Dosagem arbitrária

§ 35.1 - Para os efeitos deste Regulamento, entender-se-á por dosagem arbitrária a que for feita sem levar em conta a porcentagem de água e a graduação dos agregados.

§ 35.2 - Em qualquer concreto dosado arbitrariamente, é obrigatório um teor mínimo de 300 kg de cimento por m³ de concreto.

§ 35.3 - Em geral, os concretos à dosagem arbitrária, compor-se-ão de: 600 litros de agregado miúdo, 800 litros de agregado graúdo, 300, 350 ou 400 kgs. de cimento para um metro cúbico de concreto

§ 35.4 - Caso o volume total, obtido com as proporções do § 35.3 seja superior a um metro cúbico, diminuir-se-ão as quantidades de agregado miúdo e graúdo, conservando a relação 6/8 e sem alterar a quantidade de cimento, até obter, no máximo um metro cúbico de concreto pronto.

§ 35.5 - Os concretos de dosagem arbitrária serão, de acordo com o teor em cimento, designados respectivamente por A.300 , A.350 e A.400

C - RESISTÊNCIA DOS CONCRETOS - SOLICITAÇÕES LIMITES

§ 37 - Concretos com dosagem arbitrária

1 - As solicitações limites admissíveis para os concretos dosados arbitrariamente são as seguintes :

CONCRETO	CIMENTO NORMAL		SUPER CIMENTO	
	Pilares com cargas axiais	Em geral	Pilares com cargas axiais	Em geral
A.300	40	45	50	55
A.350	45	50	<u>55</u>	60
A.400	50	55	60	65
Unidade = kgf /cm ²				

Como a Tensão Admissível do projeto deve ter sido $\sigma_{adm.} = 45\text{kgf/cm}^2$, é provável que o concreto tenha sido executado com 350 kg de cimento por m³ de concreto.

AÇO TRACIONADO

$$\sigma_e = n \sigma_b \frac{e + \frac{d}{2} - x}{x},$$

$$\sigma_{\text{tracao aco}} = 10 \times \left(45,871 \frac{\text{kgf}}{\text{cm}^2} \right) \times \frac{6\text{cm} + 7\text{cm} - 4,37534\text{cm}}{4,37534\text{cm}} =$$

$$\sigma_{\text{tracao aco}} = \mathbf{904,2 \text{ kgf/cm}^2} < \mathbf{1200 \text{ kgf/cm}^2} \quad \mathbf{OK}$$

" **REGULAMENTO PARA CONSTRUÇÕES EM CONCRETO ARMADO**", RJ /1931.

AÇO DOCE COMUM TENSÃO DE RUPTURA : 3700 kgf.cm²

LIMITE DE ELASTICIDADE : 2400 kgf/cm² ;

Tensão admissível em serviço = 1200 kgf/cm²

AÇO COMPRIMIDO

$$\sigma_e' = n \sigma_b \frac{e - \frac{d}{2} + x}{x}.$$

$$\sigma_{\text{compressao aco}} = 10 \times \left(45,871 \frac{\text{kgf}}{\text{cm}^2} \right) \times \frac{6\text{cm} - 7\text{cm} + 4,37534\text{cm}}{4,37534\text{cm}} =$$

$$\sigma_{\text{compressao aco}} = \mathbf{353,870 \text{ kgf/cm}^2}$$

VERIFICAÇÃO DO EQUILÍBRIO DAS FORÇAS

Resultante de compressão no concreto =

$$= (b=134\text{cm}) \times (x=4,37534\text{cm}) \times (45,871 \text{ kgf/cm}^2 / 2) = \underline{13446,982\text{kgf}}$$

$$\text{Aço comprimido} = 353,870 \text{ kgf/cm}^2 \times 21\text{cm}^2 = \underline{7431,27 \text{ kgf}}$$

$$\text{COMPRESSÃO Total} = \underline{\mathbf{20878,252 \text{ kgf}}}$$

$$\text{Aço tracionado} = 904,2 \text{ kgf/cm}^2 \times 21 \text{ cm}^2 = \mathbf{18988,2 \text{ kgf}}$$

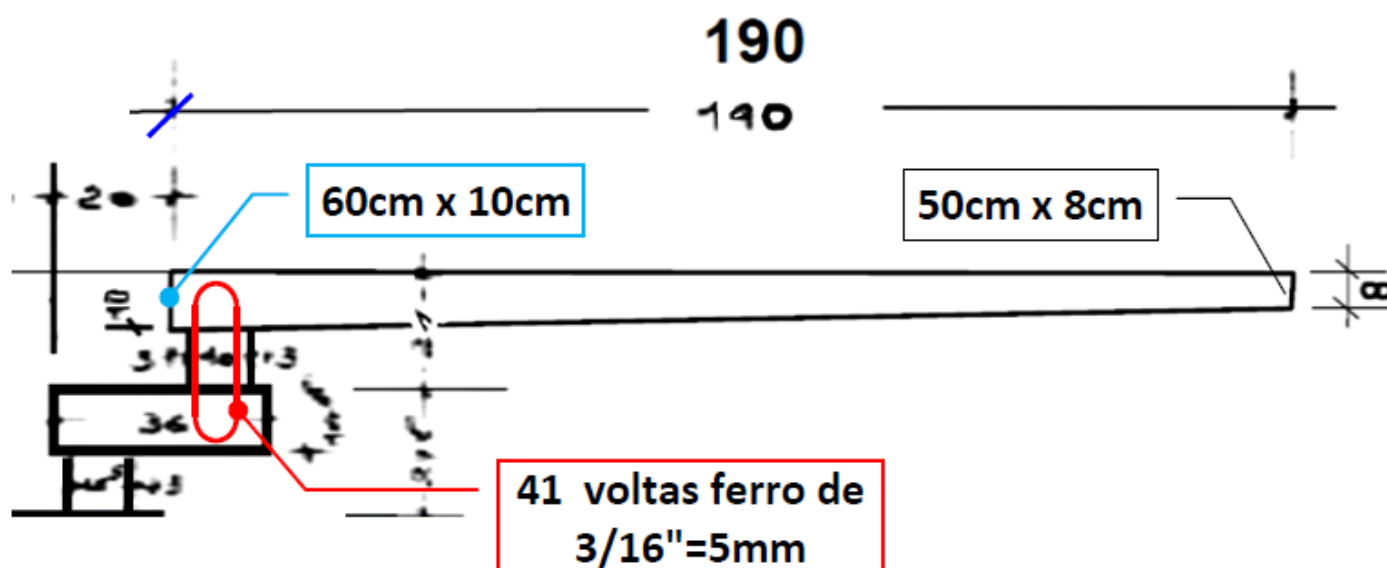
DIFERENÇA = COMPRESSÃO - TRAÇÃO

$$(\text{COMPRESSÃO} = 20878,252 \text{ kgf}) - (\text{TRAÇÃO} = 18988,2\text{kgf}) = \underline{\mathbf{1890,052 \text{ kgf}}}$$

$$\text{COMPRESSÃO N atuante} = \underline{\mathbf{1890 \text{ kgf}}} \quad \mathbf{OK}$$

EQUILÍBRIO OK

VERIFICAÇÃO DA ARMADURA DO DEGRAU MAIS ALTO



Carregamento de peso próprio

Extremo do balanço = $0,5\text{m} \times 0,08\text{m} \times 2,5\text{t/m}^3 = 0,10 \text{ t/m}$

Início do balanço = $0,6\text{m} \times 0,10\text{m} \times 2,5\text{t/m}^3 = 0,15 \text{ t/m}$

Carregamento triangular = $0,15\text{t/m} - 0,10 \text{ t/m} = 0,05 \text{ t/m}$

Carregamento retangular uniforme = $0,10 \text{ t/m}$

Momento fletor no centro do degrau =

Triangular = $(0,05\text{t/m} \times 1,9\text{m}/2) \times (1,9\text{m}/3 - 3\text{cm} - 10\text{cm}/2) =$
 $= 0,0475 \text{ t} \times 0,55\text{m} = 0,0261 \text{ t.m}$

Retangular = $(0,1\text{t/m} \times 1,9\text{m}) \times (1,9\text{m}/2 - 3\text{cm} - 10\text{cm}/2) =$
 $= 0,19 \text{ t} \times 0,87 \text{ m} = 0,165 \text{ t.m}$

Total :

Força Normal $N = 0,0475\text{t} + 0,19 \text{ t} = \underline{0,2375 \text{ t}}$

Momento fletor $M = 0,0261 \text{ t.m} + 0,165 \text{ t.m} = \underline{0,191 \text{ t.m}}$

Carregamento de obrecarga de Pessoas :

Uma pessoa na extremidade do balanço :

$N = 100 \text{ kg} = 0,1 \text{ t}$

$M = 0,1 \text{ t} \times (1,90\text{m} - 3\text{cm} - 10\text{cm}/2) = 0,182 \text{ tm}$ ou

Quatro pessoas ao longo do balanço :

$N = 4 \times 75 \text{ kgf} = \underline{0,3 \text{ t}}$

$M = 0,3 \text{ t} \times (1,90\text{m}/2 - 3\text{cm} - 10\text{cm}/2) = \underline{0,261 \text{ tm}}$

TOTAL PESO PRÓPRIO + SOBRECARGA :

$N = 0,2375 \text{ t} + 0,3 \text{ t} = \underline{\mathbf{0,5375 \text{ t}}}$

$M = 0,191 \text{ t.m} + 0,261 \text{ tm} = \underline{\mathbf{0,452 \text{ tm}}}$

DADOS DO TRAMPOLIM ITAPEMIRIM
ÚLTIMO DEGRAU : $d=10\text{cm}$; $b=54\text{ cm}$

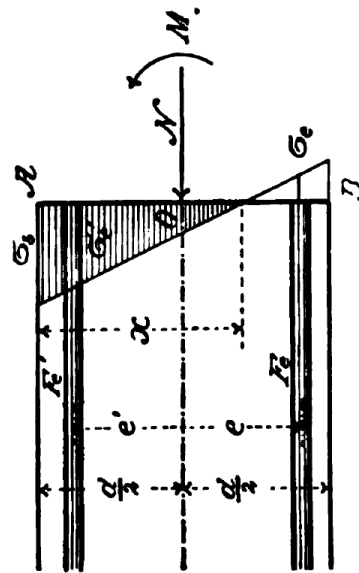


FIG. 107.

VER PLANILHA COM A GEOMETRIA DOS DEGRAUS ao final

$$N = 0,5375 \text{ t}$$

$$M = 0,452 \text{ tm}$$

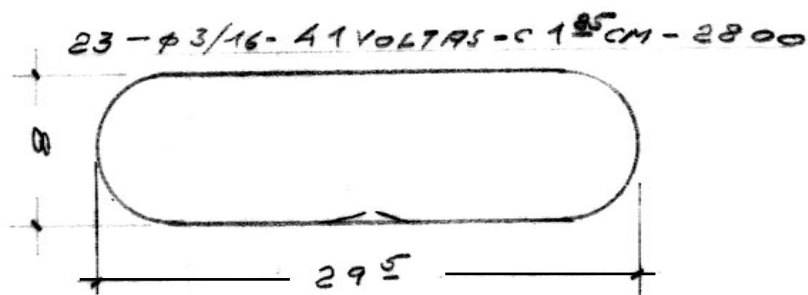
$$M/N = 0,452 \text{ tm} / 0,5375\text{t} = 0,841\text{m}$$

$$e = e' = 4\text{cm}$$

$$d = 10\text{cm}$$

$$b = 54\text{cm}$$

ARMADURA NO DEGRAU MAIS ALTO
(girada 90 graus para facilitar a leitura)



$$F_e = F_e' = 41 \text{ ferros } 5\text{mm} = 41 \times 0,2 \text{ cm}^2 = 8,2 \text{ cm}^2$$

$$n = E_{\text{aço}} / E_{\text{concreto}} =$$

$$= 2\,100\,000 \text{ kgf/cm}^2 / 200\,000 \text{ kgf/cm}^2 = 10$$

APLICANDO

$$x^3 - 3x^2 \left(\frac{d}{2} - \frac{M}{N} \right) + 12x \frac{M}{N} n \frac{F_e}{b} - 6 \frac{nF_e}{b} \left(\frac{M}{N}d + 2e^2 \right) = 0.$$

$$-3x^2 \left(\frac{d}{2} - \frac{M}{N} \right)$$

$$-3 \times \left(\frac{d}{2} - \frac{M}{N} \right) = -3 \times \left(\frac{10\text{cm}}{2} - 84,1\text{cm} \right) = -79,1\text{cm} = \underline{\underline{+237,3\text{cm}}}$$

$$+12x \frac{M}{N} n \frac{F_e}{b}$$

$$12 \times \left(\frac{M}{N} \right) \times n \times \left(\frac{F_e}{b} \right) =$$

$$+12 \times (84,1\text{cm}) \times (10) \times \left(\frac{8,2\text{cm}^2}{54\text{cm}} \right) = \underline{\underline{+1532,49\text{cm}^2}}$$

$$-6 \frac{nF_e}{b} \left(\frac{M}{N}d + 2e^2 \right)$$

$$-6 \times \left[\left(\frac{10 \times 8,2\text{cm}^2}{54\text{cm}} \right) \right] \times \left(84,1\text{cm} \times 10\text{cm} + 2 \times 4\text{cm} \times 4\text{cm} \right) =$$

$$-9,111 \times (841 + 32) = \underline{\underline{-7953,90}}$$

$$x^3 + 237,3 x^2 + 1532,49 x - 7953,90 = 0$$

RESOLVENDO COM O EXCEL

$$\underline{\underline{X = 3,3877 \text{ cm}}}$$

Tensão no concreto

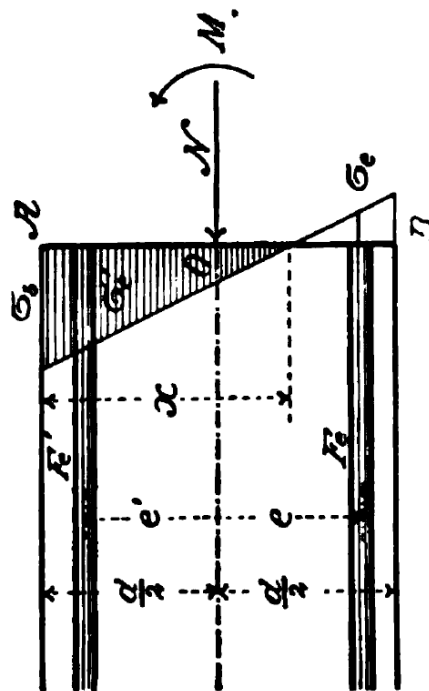


FIG. 107.

$$\sigma_b = \frac{N}{\frac{bx}{2} + \frac{nF_e}{x}(2x - d)}$$

$$N = 0,5375 \text{ t}$$

$$x = 3,3877 \text{ cm}$$

$$\begin{aligned} \text{sigma concreto} &= \frac{537,5 \text{ kgf}}{\frac{54 \text{ cm} \times 3,3877 \text{ cm}}{2} + \frac{10 \times 8,2 \text{ cm}^2}{3,3877 \text{ cm}} \times (2 \times 3,3877 - 10)} = \\ &= \frac{1890 \text{ kgf}}{91,4679 \text{ cm}^2 + 24,2052 \text{ cm} \times (-3,2246 \text{ cm}^2)} = \end{aligned}$$

$$\text{sigma concreto} = \frac{537,5 \text{ kgf}}{13,4158 \text{ cm}^2} = 40,06 \text{ kgf/cm}^2 < 45 \text{ kgf/cm}^2$$

$$\text{sigma concreto} = 45 \text{ kgf/cm}^2 = \text{limite para o CONCRETO A.350}$$

AÇO TRACIONADO

$$\sigma_e = n \sigma_b \frac{e + \frac{d}{2} - x}{x},$$

$$\text{sigma tracao aco} = 10 \times \left(40,06 \frac{\text{kgf}}{\text{cm}^2} \right) \times \frac{4\text{cm} + 5\text{cm} - 3,3877 \text{ cm}}{3,3877 \text{ cm}} =$$

$$\text{sigma tracao aco} = \mathbf{663,66 \text{ kgf/cm}^2} < \mathbf{1200 \text{ kgf/cm}^2} \quad \mathbf{OK}$$

" REGULAMENTO PARA CONSTRUÇÕES EM CONCRETO ARMADO", RJ /1931.

AÇO DOCE COMUM TENSÃO DE RUPTURA : 3700 kgf.cm²

LIMITE DE ELASTICIDADE : 2400 kgf/cm² ;

Tensão admissível em serviço = 1200 kgf/cm²

AÇO COMPRIMIDO

$$\sigma_e' = n \sigma_b \frac{e - \frac{d}{2} + x}{x}.$$

$$\text{sigma compressao aco} = 10 \times \left(40,06 \frac{\text{kgf}}{\text{cm}^2} \right) \times \frac{4\text{cm} - 5\text{cm} + 3,3877\text{cm}}{3,3877\text{cm}} =$$

$$\text{sigma compressao aco} = \mathbf{282,349 \text{ kgf/cm}^2}$$

VERIFICAÇÃO DO EQUILÍBRIO DAS FORÇAS

Resultante de compressão no concreto =
= (b=54cm) x (x=3,3877cm) x (40,06 kgf/cm² / 2) = 3664,2040kgf

Aço comprimido = 282,349kgf/cm² x 8,2cm² = 2315,2618kgf

COMPRESSÃO Total = 5979,4658 kgf

Aço tracionado = 663,66kgf/cm² x 8,2 cm² = 5442,012 kgf

DIFERENÇA = COMPRESSÃO - TRAÇÃO
(COMPRESSÃO =5979,4658 kgf) - (TRAÇÃO =5442,012 kgf) = 537,4538 kgf

COMPRESSÃO N atuante = 0,5375 t = 537,5 kgf

EQUILÍBRIO OK

GEOMETRIA DO TRAMPOLIM								
Elemento			Bx (cm)	Hy (cm)	Lz (cm)	peso (ton)	X (cm)	momento (ton. m)
DEGRAU		1	14	11	134	0,05159	0	0,00
	PISO	2	39,5	14	140	0,19355	9,75	0,02
DEGRAU		3	13,5	11	124	0,046035	19,75	0,01
	PISO	4	39	13,5	130	0,1711125	29,5	0,05
DEGRAU		5	13	11	114	0,040755	39,5	0,02
	PISO	6	38,5	13	120	0,15015	49,25	0,07
DEGRAU		7	12,5	11	104	0,03575	59,25	0,02
	PISO	8	38	12,5	110	0,130625	69	0,09
DEGRAU		9	12	11	94	0,03102	79	0,02
	PISO	10	37,5	12	100	0,1125	88,75	0,10
DEGRAU		11	11,5	11	84	0,026565	98,75	0,03
	PISO	12	37	11,5	90	0,0957375	108,5	0,10
DEGRAU		13	11	11	74	0,022385	118,5	0,03
	PISO	14	36,5	11	80	0,0803	128,25	0,10
DEGRAU		15	10,5	11	64	0,01848	138,25	0,03
	PISO	16	36	10,5	70	0,06615	148	0,10
DEGRAU		17	10	11	54	0,01485	158	0,02
	TRAMPOLIM	18	200	10	60	0,3	255	0,77
	TOTAL					1,59 ton		1,58 ton.m
	MAIS	UM HOMEM				0,1 ton	3,4m	0,34tm
	OU							
	MAIS	4 PESSOAS				0,3ton	2,5m	0,75tm
				TOTAL		1,89 ton		2,33t.m

Na Universidade do Estado de Goiás, U.E.G, foram medidas as forças de saída do pulo em 15 saltadores, (ver adiante, na página 62)

Considerando um saltador na ponta do balanço com a máxima força de saída do pulo medida na U.E.G, igual a 199 kgf, obtemos o momento :

$M_{\text{saltador}} = 199 \text{ kgf} \times 3,4\text{m} = 0,68 \text{ t.m}$, menor que o momento fletor adotado, considerando 4 pessoas estaticamente situadas no trampolim, como indicado acima.

Comentário do Eng. Gilberto Couri

" Trabalhei na SEEBLA (Serviços de Engenharia Emílio Baumgart Ltda) por alguns anos, diretamente com o Prof. Arthur Eugenio Jermann.

Lá, tínhamos, à época, o arquivo do antigo Escritório Técnico Emílio Baumgart (que antecedeu a SEEBLA) e guardavam desde o projeto nº 1.

Sei que a escada foi demolida.

A inércia variável mais o detalhe da ferragem faziam com que a amplitude do conjunto (escada e a prancha), tudo em concreto, fosse similar à da madeira (só a prancha).

Mas havia uma condição, ninguém podia ficar sobre a escada enquanto um saltador estivesse na prancha, pois atrapalharia a deformação do conjunto escada-prancha de concreto , além de desequilibrá-lo.

A ferragem era detalhada em estribos sucessivos, que pegavam os espelhos e os pisos.

Gilberto Couri "

Comentário do Eng. E.C.S.Thomaz

Em abril de 1923 fora mostrada, no Magazine "Eu sei tudo" / RJ, uma escada na Siderúrgica Creusot, na França, similar a esse trampolim. Baumgart, no seu trampolim, eliminou a viga inclinada e a estrutura ficou apenas com os degraus.



PÁGINA EM BRANCO

CÁLCULO DA FLECHA

Segundo o Prof. Telemaco van Langendonck :



Flecha das vigas, em serviço :

“ A deformação por flexão das peças de concreto armado (CA50) pode se calculada com razoável precisão tomando-se o valor do momento de inércia calculado no estágio II considerando o módulo de elasticidade $E_a=205$ GPa do aço e o módulo E_c real do concreto empregado.”

14.2.2 Estádio II

No estágio II o concreto tracionado é desprezado, pois ele está fissurado (Figura 14.2).

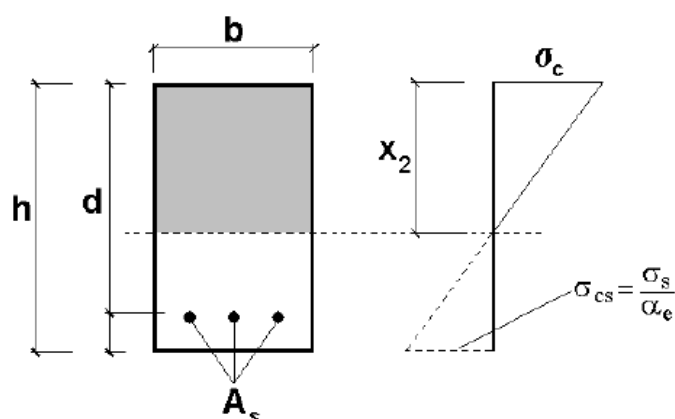


Figura 14.2 – Seção retangular no Estádio II

Com procedimento análogo ao do estágio I, tem-se para seção retangular no estágio II (Figura 14.2):

$$M_{LN} = b \cdot x \cdot \frac{x}{2} - (\alpha_e - 1) \cdot A_s \cdot (d - x) = 0 \rightarrow x_2$$

Portanto, a posição da linha neutra x_2 é obtida por meio da equação:

$$\frac{b}{2} \cdot x_2^2 + (\alpha_e - 1) \cdot A_s \cdot x_2 - (\alpha_e - 1) \cdot A_s \cdot d = 0$$

Momento de inércia I_2 :

$$I_2 = \frac{b \cdot x_2^3}{12} + b \cdot x_2 \cdot \left(\frac{x_2}{2}\right)^2 + (\alpha_e - 1) \cdot A_s \cdot (d - x_2)^2$$

ou

$$I_2 = \frac{b \cdot x_2^3}{3} + (\alpha_e - 1) \cdot A_s \cdot (d - x_2)^2$$

SEGUE

Cálculo de x no estágio II

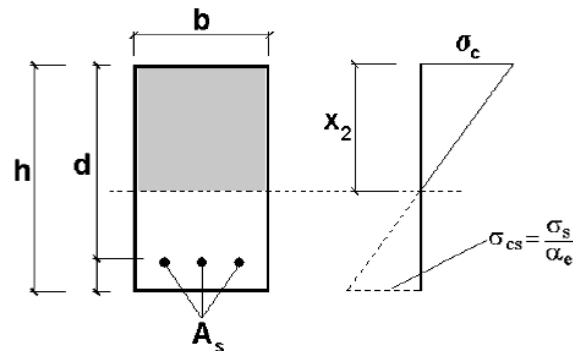


Figura 14.2 – Seção retangular no Estádio II

Portanto, a posição da linha neutra x_2 é obtida por meio da equação:

$$\frac{b}{2} \cdot x_2^2 + (\alpha_e - 1) \cdot A_s \cdot x_2 - (\alpha_e - 1) \cdot A_s \cdot d = 0$$

No balanço do trampolim :

8cm

$$A_s = 21 \text{ barras } 6,3\text{mm} = 21 \times 0,31 \text{ cm}^2 = 6,51 \text{ cm}^2$$

$$b = 60 \text{ cm}$$

$$h = 10 \text{ cm}$$

$$d = 9 \text{ cm}$$

$$\alpha = E_a/E_c = 2 \ 100 \ 000 / 250 \ 000 = 8,4$$

$$\frac{60\text{cm}}{2} \times x^2 + (8,4 - 1) \times 6,51\text{cm}^2 \times x - (8,4 - 1) \times 6,51\text{cm}^2 \times 9\text{cm} = 0$$

$$30(\text{cm}) x^2 + 48,174(\text{cm}) x - 433,566 = 0$$

$$30x^2 + 48,174 x - 433,566 = 0$$

$$x = 3,082 \text{ cm}$$

$$I_2 = \frac{b \cdot x_2^3}{3} + (\alpha_e - 1) \cdot A_s \cdot (d - x_2)^2$$

$$I_2 = \frac{60\text{cm} \times (3,082\text{cm})^3}{3} + (8,4 - 1) \times 6,51\text{cm}^2 \times (9\text{cm} - 3,082\text{cm})^2$$

$$I_2 = 878,252 + 1687,185 = 2565,437\text{cm}^4$$

$$I_1 = 60\text{cm} \times (10\text{cm})^3 / 12 = 5000\text{cm}^4$$

$$I_2 / I_1 = 2565,45 / 5000 = \mathbf{0,51}$$

PARA USAR O FTOOL , QUE CALCULA AUTOMATICAMENTE A INÉRCIA A PARTIR DAS DIMENSÕES **b** e **h** , usaremos uma largura reduzida =

No balanço $b_i = \mathbf{0,51} \times 60 = 31\text{ cm}$

A largura do balanço varia de 60cm a 50cm

Considerarei 3 trechos retos com a largura média no trecho :
 no primeiro trecho $b_{mi} = 0,51 \times 58,5\text{ cm} = 31\text{cm}$ $h_m = 9,7\text{ cm}$
 no segundo trecho $b_{mi} = 0,51 \times 55,5\text{ cm} = 28\text{cm}$ $h_m = 9,0\text{ cm}$
 no terceiro trecho $b_{mi} = 0,51 \times 52\text{cm} = 26\text{cm}$ $h_m = 8,4\text{ cm}$

**COMO OS DEGRAUS E OS ESPELHOS TEM FORTE ARMADURA DUPLA
 USAREMOS A INERCIA NO ESTÁDIO I , QUE É A CALCULADA PELO FTOOL.**

NOS DEGRAUS 1 ao 3 $b_i = 124\text{cm}$ $h = 13,5\text{ cm}$

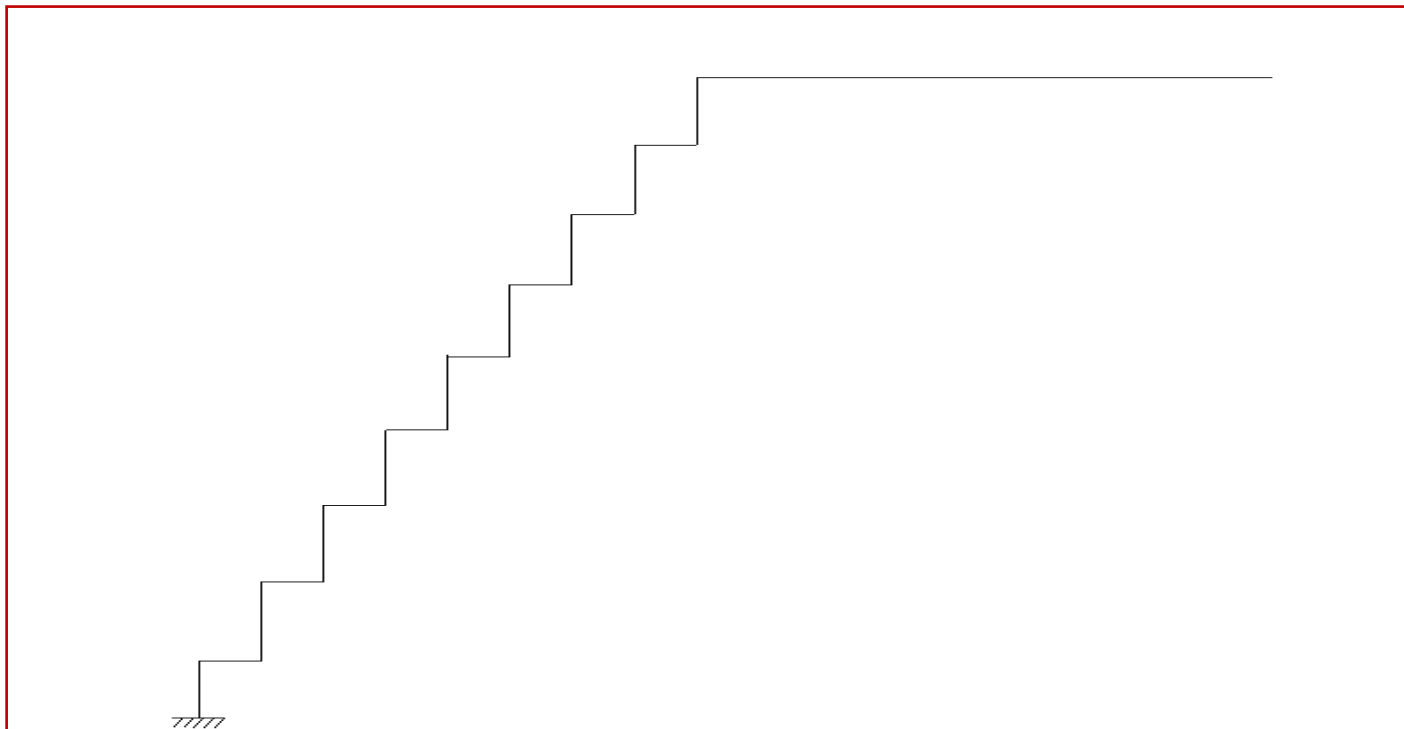
NOS DEGRAUS 4 ao 6 $b_i = 94\text{ cm}$ $h = 12,0\text{ cm}$

NOS DEGRAUS 7 ao 9 $b_i = 64\text{ cm}$ $h = 10,5\text{ cm}$

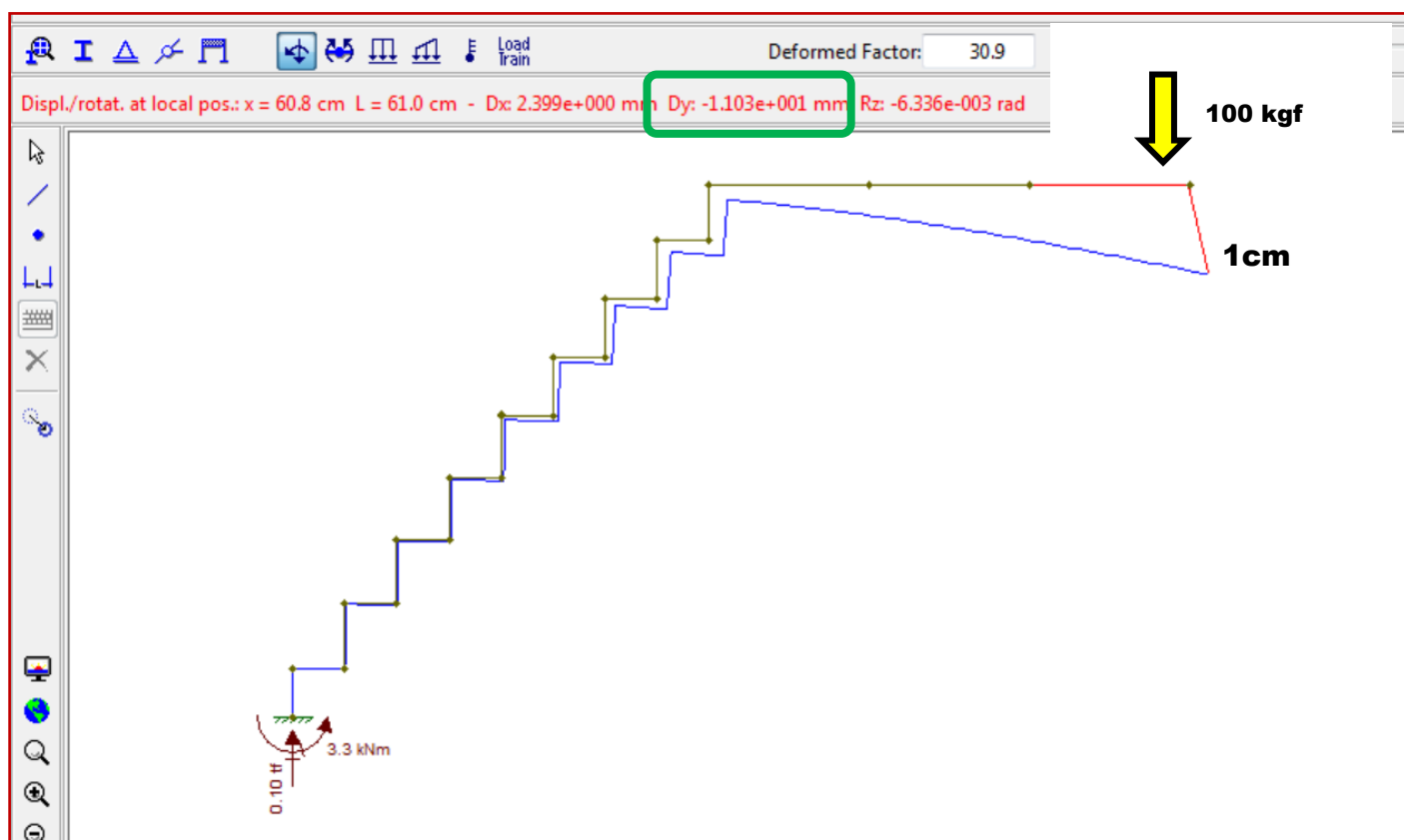
A ANÁLISE ESTRUTURAL FOI FEITA USANDO O PROGRAMA FTOOL
DO PROF. LUIZ FERNANDO MARTHA DA PUC/RJ

<http://webserver2.tecgraf.puc-rio.br/~lfm/>

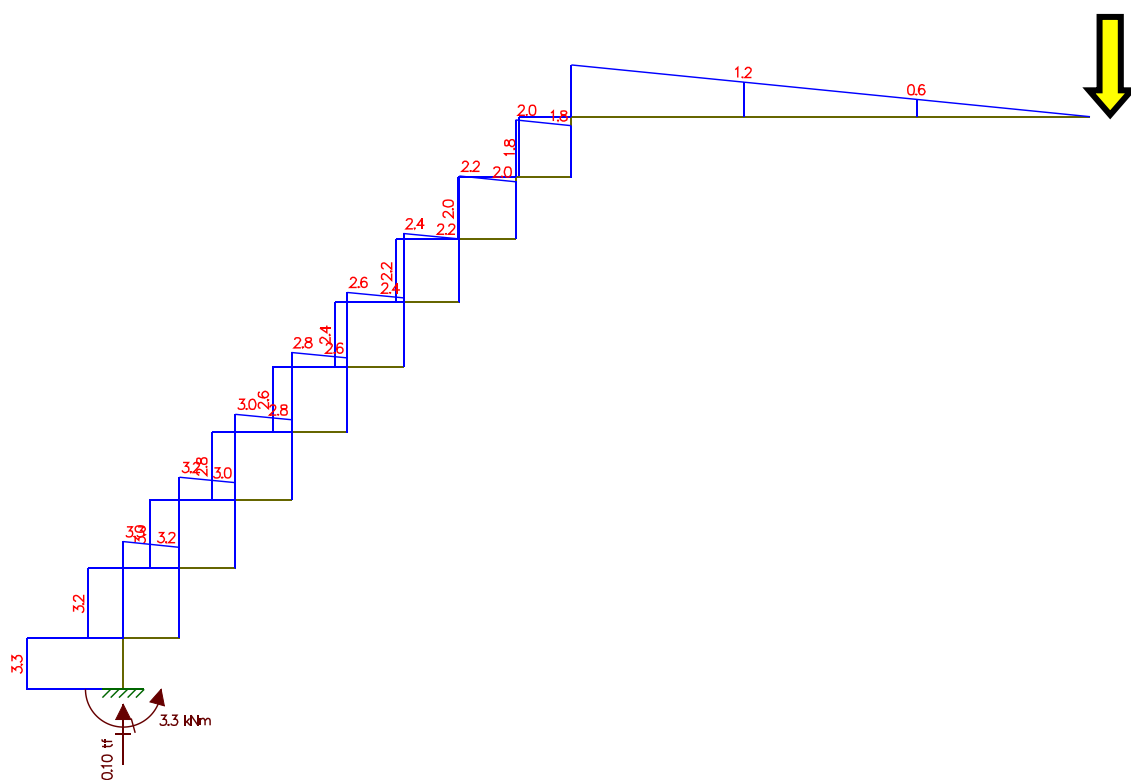
GEOMETRIA



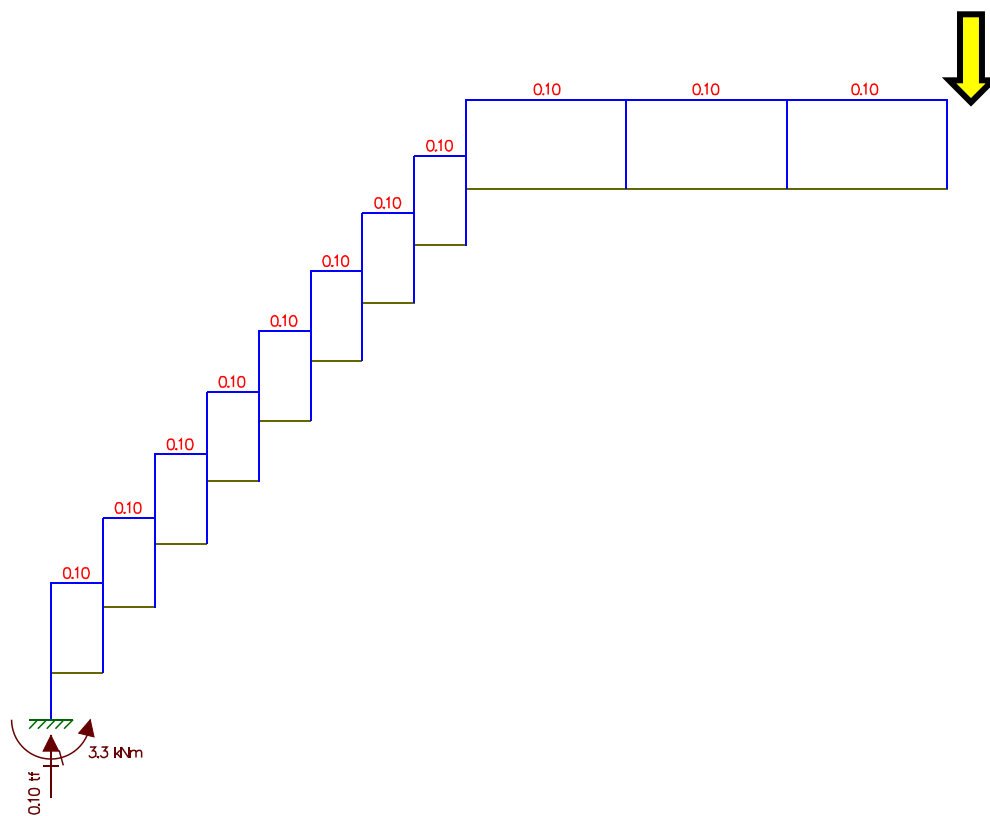
FLECHA PARA UM SALTADOR DE 100 kg estático



MOMENTO FLETOR



FORÇA CORTANTE



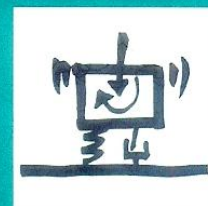
+++

PÁGINA EM BRANCO

VIBRATION PROBLEMS IN STRUCTURES

PRACTICAL GUIDELINES

HUGO BACHMANN
 WALTER J. AMMANN
 FLORIAN DEISCHL
 JOSEF EISENMANN
 INGOMAR FLOEGL
 GERHARD H. HIRSCH
 GÜNTER K. KLEIN
 GÖRAN J. LANDE
 OSKAR MAHREHOLTZ
 HANS G. NATKE
 HANS NUSSBAUMER
 ANTHONY J. PRETLOVE
 JOHANN H. RAINER
 ERNST-ULRICH SAEMANN
 LORENZ STEINBEISSER



BIRKHÄUSER

1.5 High-diving platforms

H. Bachmann, A.J. Pretlove, J.H. Rainer

1.5.1 Problem description

High-diving platforms in open air or indoor swimming pools can be affected by vibrations if the platforms have not been designed for dynamic forces [1.12]. The vibrations are mainly caused by the athlete through impulsive action immediately before or at take-off.

1.5.2 Dynamic actions

A major distinction has to be made as to whether or not a springboard for figure diving is mounted on the platform.

Normal high diving is done from a rigid concrete platform slab with or without a running start. For jumping off without running, the dynamic force consists of a single impulse. Jumping off after running activates additional impulses.

For figure diving a relatively soft springboard is mounted on the platform. The springboard flexibility results in larger amplitudes attained by the centre of mass of the athlete's body when he or she gains momentum by jumping on the spot or by running. Compared with a rigid platform slab, impulses on the springboard are significantly larger.

For design purposes the forcing function need not be known. A simple distinction between a platform with a rigid slab and a platform with mounted springboard is sufficient.

1.5.3 Structural criteria

a) Natural frequencies

High-diving platforms designed only for static loads often have natural frequencies between 2 and 3 Hz [1.12], [1.11]. Athletes can excite such platforms to excessive vibrations.

The following vibration patterns are possible:

- swaying of the support column (in a direction not necessarily coincident with the take-off direction)
- rigid-body motion of the platform (similar effects)
- vibration of the platform slab (particularly unpleasant as the athlete may be given an unwanted spin).

b) Damping

In the case of high-diving platform structures, material damping of the structure itself can be augmented by considerable energy radiation to the soil (see Appendix C). This may be true when the foundation of the platform structure stands on rather soft soil and can rotate, (i.e. it

is not connected to a basin or its foundation) or when the shaft above the foundation is embedded over a certain length of soil. Then the vibration deformation of the structure can lead to considerable energy radiation into the soil. However, most high-diving platform structures have a relatively low equivalent viscous damping ratio ζ as shown in Table 1.3.

Construction type	damping ratio ζ		
	min.	mean	max.
Reinforced concrete (~ uncracked or only few cracks)	0.008	0.012	0.016

Table 1.3: Common values of damping ratio ζ for high-diving platform structures

1.5.4 Effects

Strong support column vibrations irritate the athlete and hamper his or her performance. In extreme cases, the high-diving platform has to be strengthened or totally rebuilt [1.11].

Vibrations mainly affect the serviceability of the platform. Problems of fatigue or impending failure are rarely relevant.

1.5.5 Tolerable values

The definition of upper limits of velocities or accelerations is impractical as the vibrations are transient and their direction is also of significance. It has been found more useful to comply with certain frequency and stiffness criteria (see paragraph below).

1.5.6 Simple design rules

A high-diving platform must fulfill the following two types of criteria [1.12]:

- stiffness criteria to be checked with relatively simple static calculations
- frequency criteria corresponding to high tuning of the structure and therefore requiring a frequency computation.

a) Stiffness criteria

The spatial vector displacement of the front edge of the platform caused by spatial static force with $2F_x = F_y = F_z = 1$ kN according to Figure 1.9 must remain

$$\delta(F_x, F_y, F_z) \leq 1 \text{ mm} \quad (1.10)$$

and the lateral front displacement alone must be

$$\delta_x = 0.5 \cdot \delta \leq 0.5 \text{ mm.} \quad (1.11)$$

The stiffness criteria are particularly stringent. Practical experience shows that reinforced-concrete platforms can be assumed to maintain their uncracked stiffness in bending as well as

torsion. The listed bounds were derived from approved high-performance platforms and represent competition standards for normal high diving and for figure diving. For less professional demands in recreational indoor or outdoor swimming facilities, these bounds could well be relaxed.

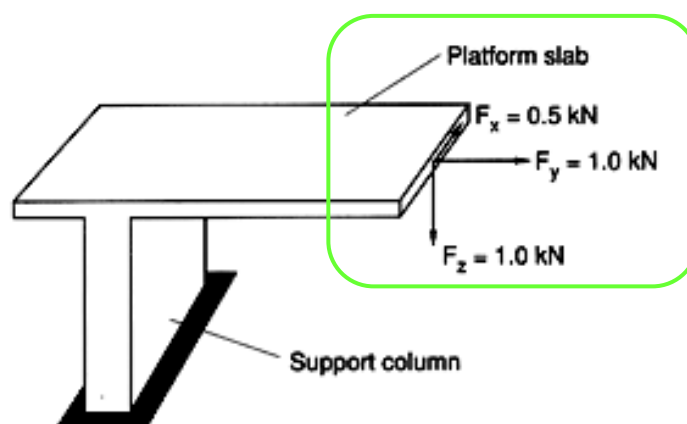


Figure 1.9: High-diving platform with spatial static load [1.12]

b) Frequency criteria

The frequency bounds to be observed are listed in Table 1.4. They concern support column sway, rigid-body motion and platform slab vibration. A major distinction is to be made when a springboard for figure diving is mounted on the platform. As described before, rhythmic jumping on the springboard contributes much to the excitation of the platform, so that stricter frequency bounds apply.

Frequency bounds	without spring board	with spring board
Support column vibrations (all fundamental modes in longitudinal and lateral sway and in twist)	$f_1 \geq 3.5 \text{ Hz}$	$f_1 \geq 5.0 \text{ Hz}$
Rigid-body vibration (flexibility of foundation)	$f_1 \geq 7.0 \text{ Hz}$	$f_1 \geq 10.0 \text{ Hz}$
Slab vibration	$f_1 \geq 10.0 \text{ Hz}$	$f_1 \geq 10.0 \text{ Hz}$

Table 1.4: Recommended minimum frequencies for high-diving platforms in swimming pools

1.5.7 More advanced design rules

No design rules can be recommended other than those given above.

1.5.8 Remedial measures

Inadequate high-diving platforms can be strengthened with the objective of attaining the frequency and stiffness criteria given in Section 1.5.6. In some cases a tuned vibration absorber can be installed [1.13]. An increase of inherent damping or other measures for improvement are generally difficult to put into practice.

References to Chapter 1

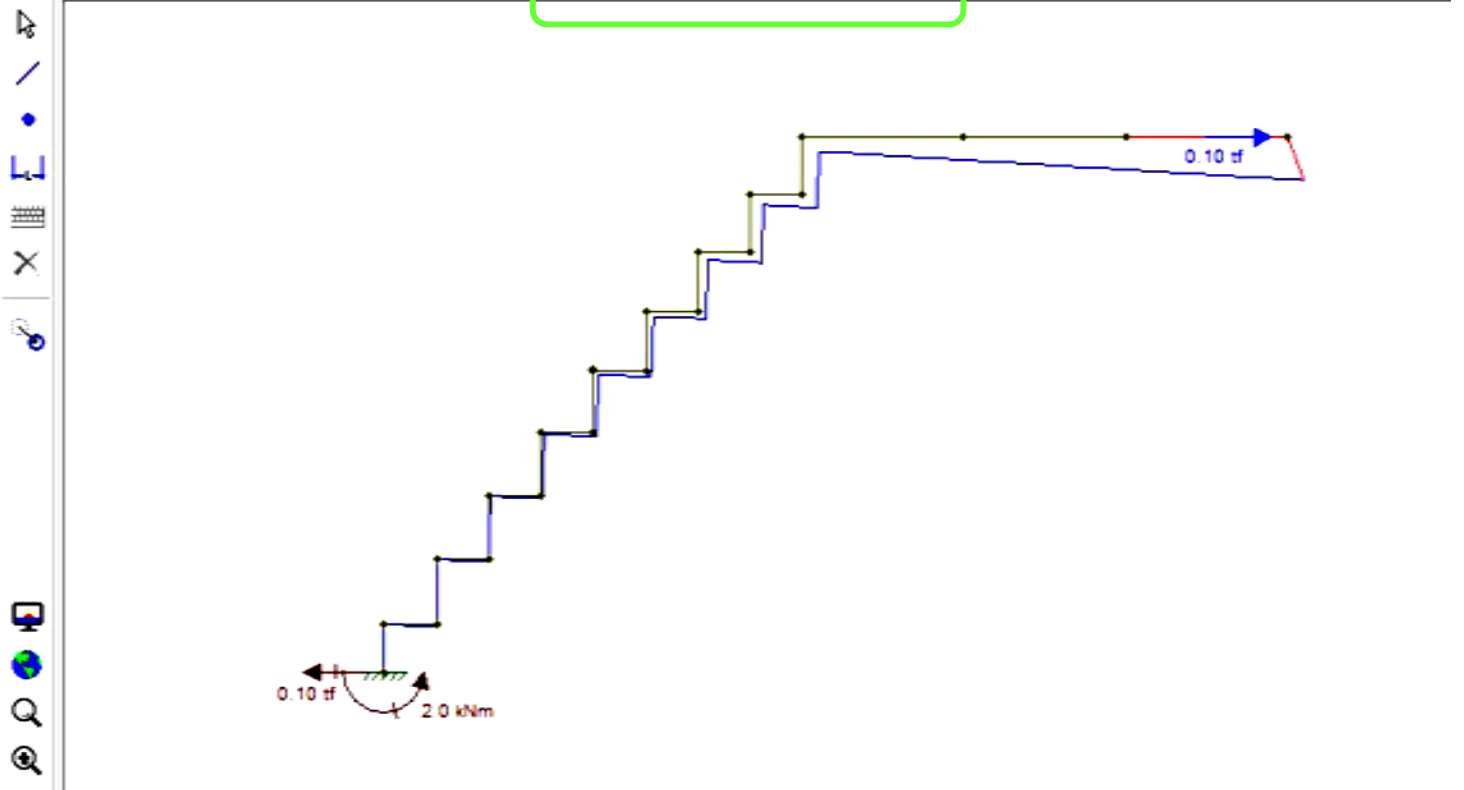
- [1.1] Tilly G.P., Cullington D.W., Eyre R.: "Dynamic Behaviour of Footbridges". Surveys S-26/84 of the International Association of Bridge and Structural Engineering (IABSE), 1984.
- [1.2] Wiss J.F., Parmelee R.A.: "Human Perception of Transient Vibrations". Proceedings of the American Society of Civil Engineers (A.S.C.E.), 100, ST4, 773, 1974.
- [1.3] Rainer J.H., Pernica G., Allen D.E.: "Dynamic Loading and Response of Footbridges". Canadian Journal of Civil Engineering, 15(1), 66, 1988.
- [1.4] Jones R.T., Pretlove A.J., Eyre R.: "Two Case Studies in the Use of Tuned Vibration Absorbers on Footbridges". The Structural Engineer, 59B, 27, 1981.
- [1.5] Allen D.E., Rainer J.H. and Pernica G.: "Vibration Criteria for Long-Span Concrete Floors". In "Vibrations of Concrete Structures", Special Publication SP-60, p. 67-78, American Concrete Institute, Detroit, Michigan, 1979.
- [1.6] Allen D.E. and Rainer J.H.: "Vibration Criteria for Long-Span Floors", Canadian Journal of Civil Engineering, 3, (2) June 1976, p.165-173, 1976.
- [1.7] Murray T.M.: "Acceptability Criterion for Occupancy-Induced Floor Vibrations". Engineering Journal, American Institute for Steel Construction, 18(2), 1981, p. 62-70, 1981.
- [1.8] Ohlsson S.: "Springiness and Human Induced Floor Vibration - A Design Guide". Document D12:1988, Swedish Council for Building Research, Stockholm, 1988.
- [1.9] Wyatt T.A.: "Design Guide on the Vibration of Floors". Publication 076, The Steel Construction Institute (SCI), Sunningdale, Berkshire U.K., 1989.
- [1.10] Allen D.E., Murray T.M.: "Design Criterion for Vibrations Due to Walking". Engineering Journal, American Institute of Steel Construction (AISC), Vol. 30, No. 4, 1993, pp. 117 - 129.
- [1.11] Bachmann H., Ammann W.: "Vibrations in structures - Induced by Man and Machines". Structural Engineering Documents No.3e, International Association for Bridge and Structural Engineering (IABSE), Zürich, 1987.
- [1.12] Mayer H.: "Schwingungsverhalten von Sprungtürmen in Frei- und Hallenbädern" ("Vibrational Behaviour of High-Diving Platforms in Outdoor and Indoor Swimming Pools"). Jahresbericht der Höheren Technischen Lehranstalt (HTL), Brugg-Windisch, Schweiz, 1970/1971.
- [1.13] Bachmann H.: "Beruhigung eines Sprungturms durch einen Horizontaltilger" ("Vibration upgrading of a High-Diving Platform by means of a Horizontally Acting Tuned Vibration Absorber"). Schweizer Ingenieur und Architekt Nr. 21/1994, Zürich, 1994.
- [1.14] Tilly G.P. (ed.): "Dynamic Behaviour of Concrete Structures". RILEM 65 MDB Committee, Elsevier, 1986.

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TESTE DA RIGIDEZ PARA UMA PLATAFORMA (BACHMANN)

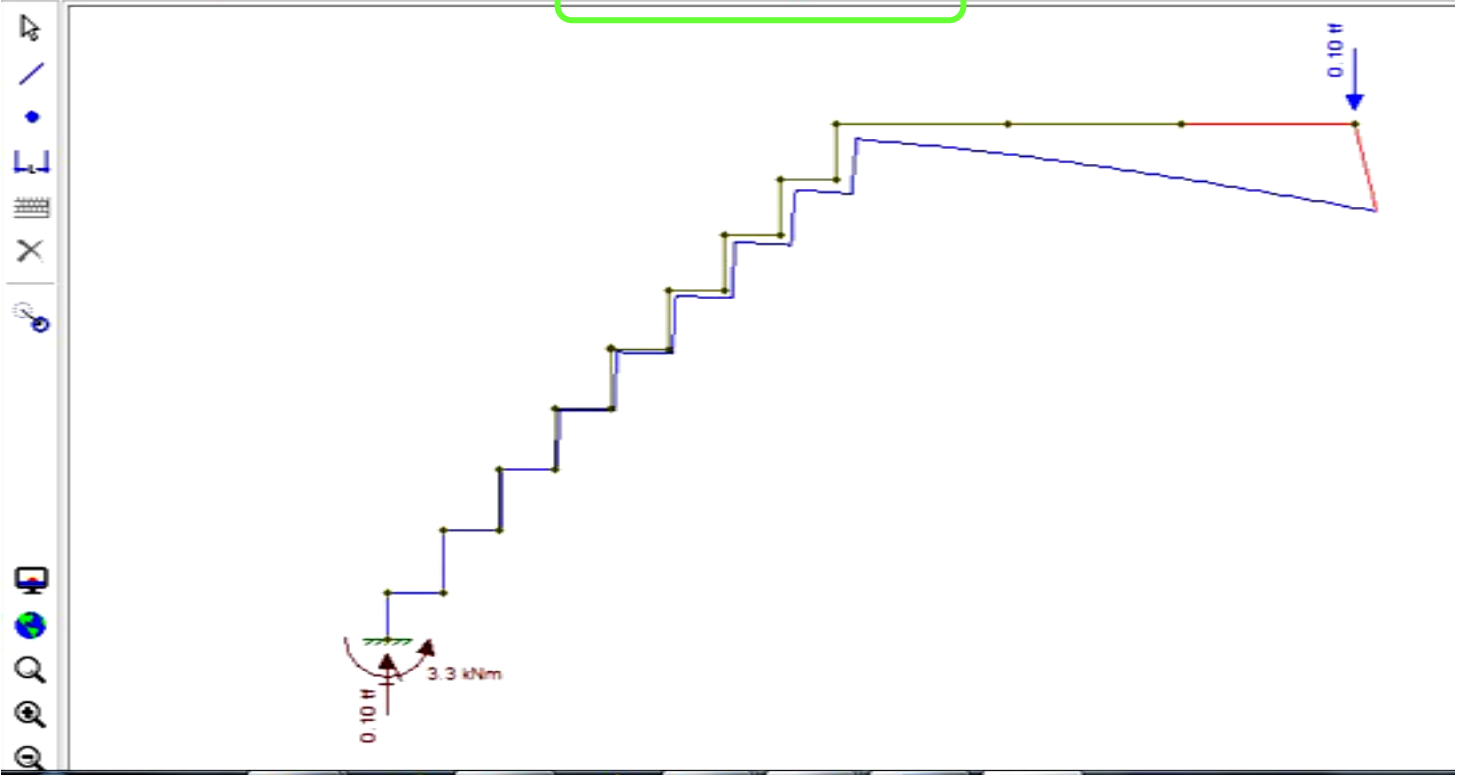
Deformações para $F_X = 100 \text{ kgf}$

Displ./rotat. at local pos.: x = 61.0 cm L = 61.0 cm - Dx: 9.786e-001 mm Dy: -2.399e+000 mm Rz: -8.922e-004 rad

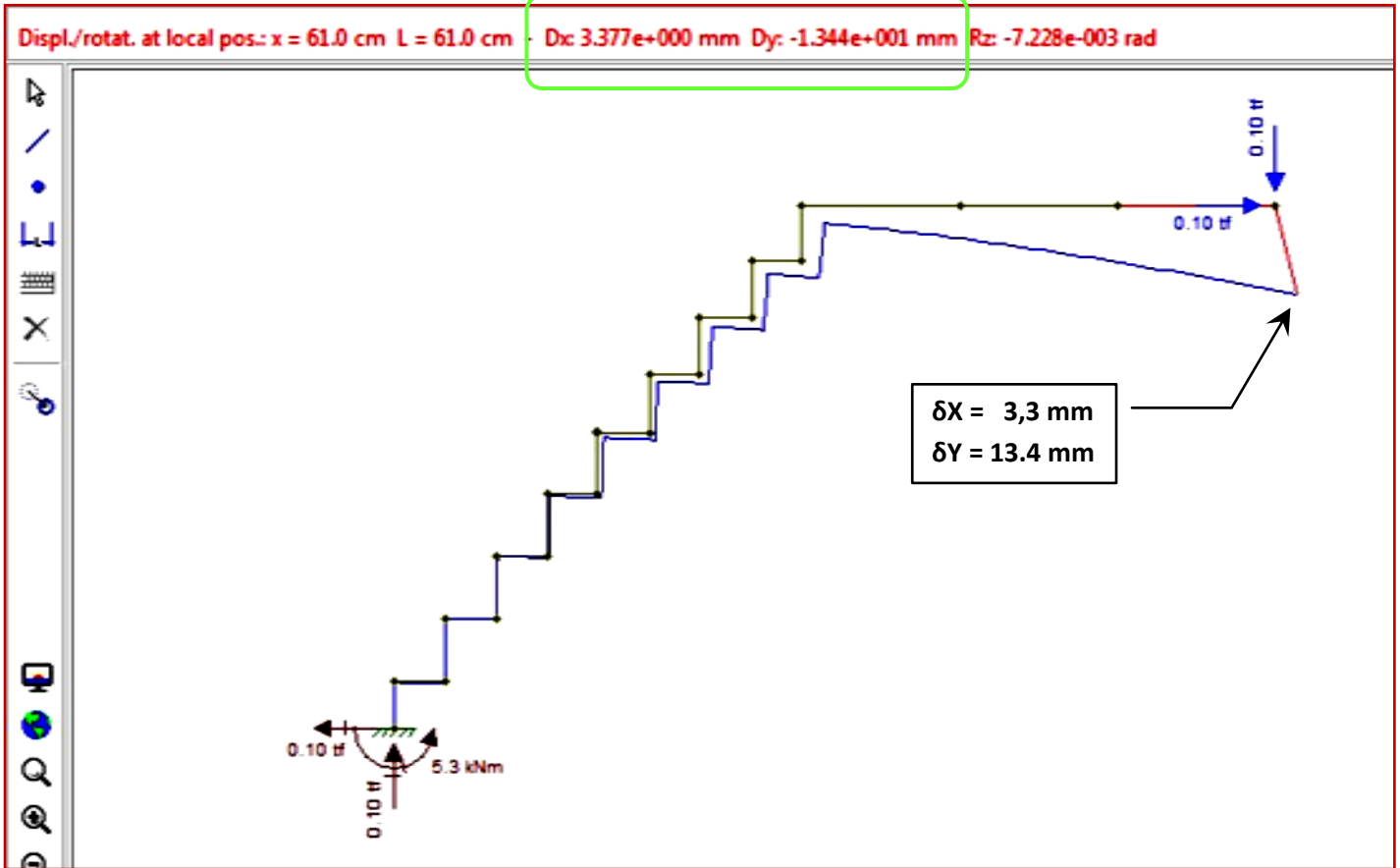


Deformações para $F_Y = 100 \text{ kgf}$

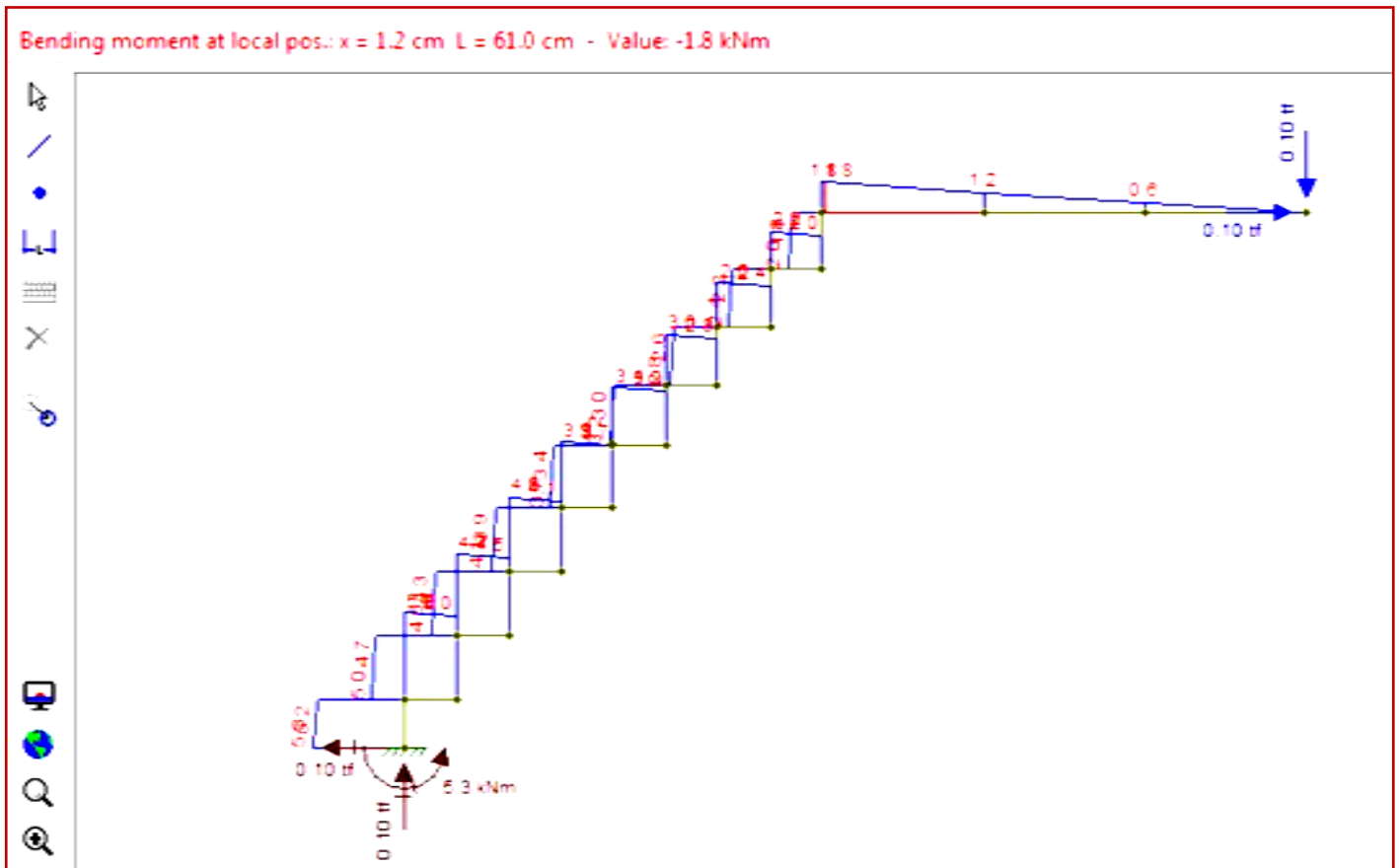
Displ./rotat. at local pos.: x = 61.0 cm L = 61.0 cm - Dx: 2.399e+000 mm Dy: -1.104e+001 mm Rz: -6.336e-003 rad



Deformações para $F_X = 100 \text{ kgf} + F_Y = 100 \text{ kgf}$



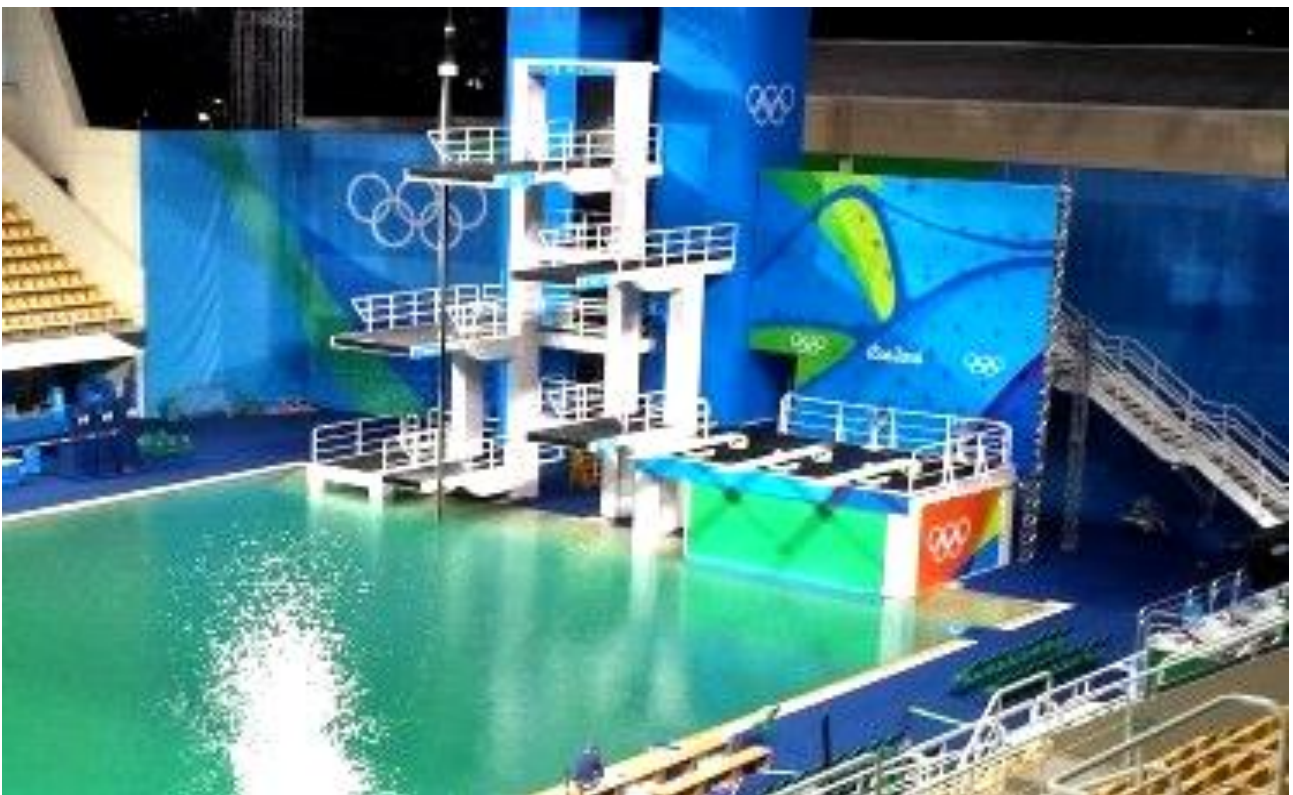
As deformações são muito maiores que os limites para uma plataforma rígida de saltos. ($\delta X < 1 \text{ mm}$ e $\delta Y < 1 \text{ mm}$). Mostram pois um trampolim flexível.



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FORÇAS NO TRAMPOLIM

Standing, walking, running, and jumping on a force plate

Rod Cross , Physics Department, University of Sydney, Australia - 1998

<http://www.physics.usyd.edu.au/~cross/PUBLICATIONS/6.%20StandingForcePlate.PDF>

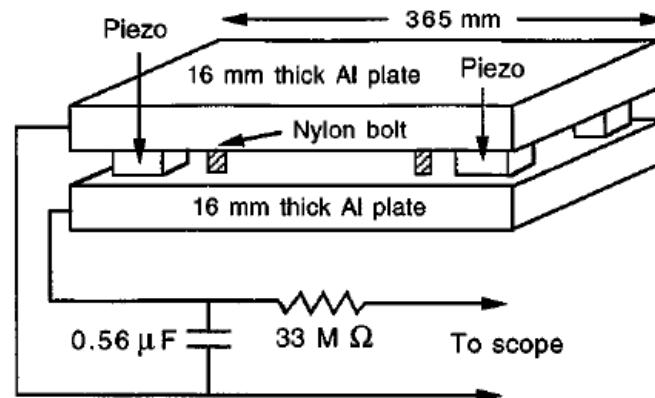


Fig. 1. The homemade force plate. A thin sheet of rubber was glued to the outer surfaces of both plates to prevent slipping.

III. STANDING AND JUMPING WAVE FORMS

If one stands with both feet on a force plate, it will register a force $F=M.g$.

If the center of mass (CM) is then lowered by bending the knees, the force does not remain equal to $M.g$.

Instead, the force decreases ($F<M.g$) and then increases ($F>M.g$) before settling back to $M.g$.

Alternatively, if one steps onto the force platform in a crouching position (agachado) and then stands up straight, the result shown in **Fig. 2** is obtained.

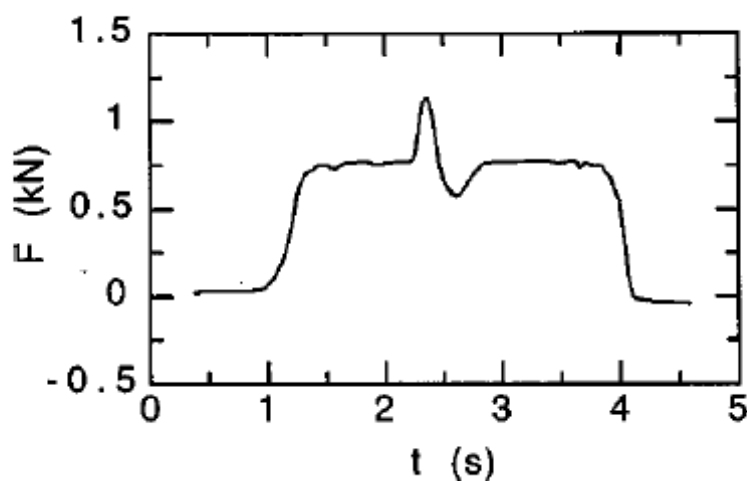


Fig. 2. The wave form observed when a person steps onto the force plate, in a crouching position, then stands up straight, then steps off the force plate.

In raising the CM, the CM starts with zero speed, accelerates to finite speed, then decelerates to a new resting position. During this maneuver, the plate registers a force F given by $F - M.g = M.a$, where a is the acceleration of the CM vertically upward.

Figure 3 shows the wave form observed when jumping off the floor and dropping onto the force plate from a height of a few cm, landing with both feet simultaneously. The force rises rapidly to a value significantly larger than Mg . The magnitude of the impact force can be reduced by allowing the knees to bend more on contact, or increased by keeping the legs straight.

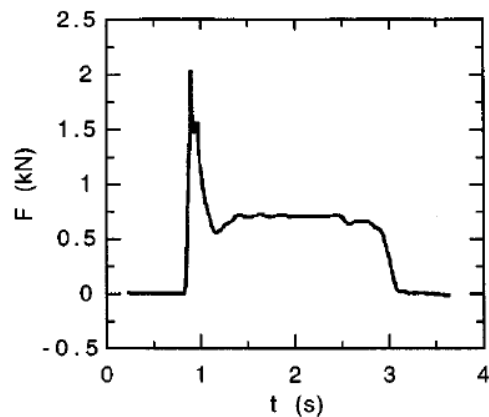


Fig. 3. The wave form observed when jumping off the floor onto the force plate and then stepping off the plate.

The CM has a negative velocity at contact, decreasing rapidly to zero with a slight positive velocity overshoot due to flexure of the knees.

The initial acceleration is therefore large and positive in a direction vertically up-ward, so $F > Mg$ initially. This is a good example where a deceleration in one direction can usefully be interpreted as an acceleration in the opposite direction.

The magnitude of the force is easily calculated, using estimates of the initial velocity and the time taken to come to rest.

Figure 4 shows the wave form that results when jumping off the platform from a standing start.

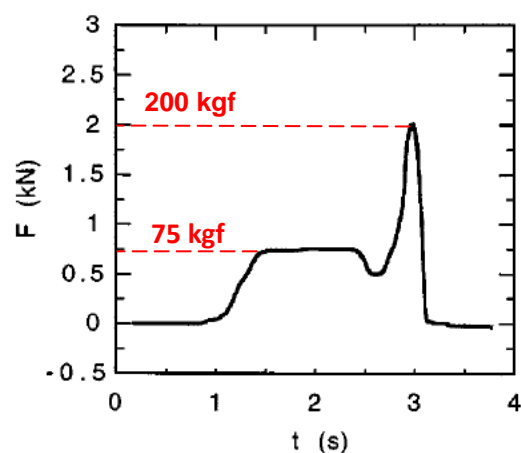


Fig. 4. The wave form observed when stepping onto and then jumping off the force plate.

The initial response corresponds to a slight lowering of the CM, in preparation for the jump. The impulse is simply related to the height of the jump, which was about 3 cm in this case.

Força máxima no piso = 2,7 x Peso do Saltador

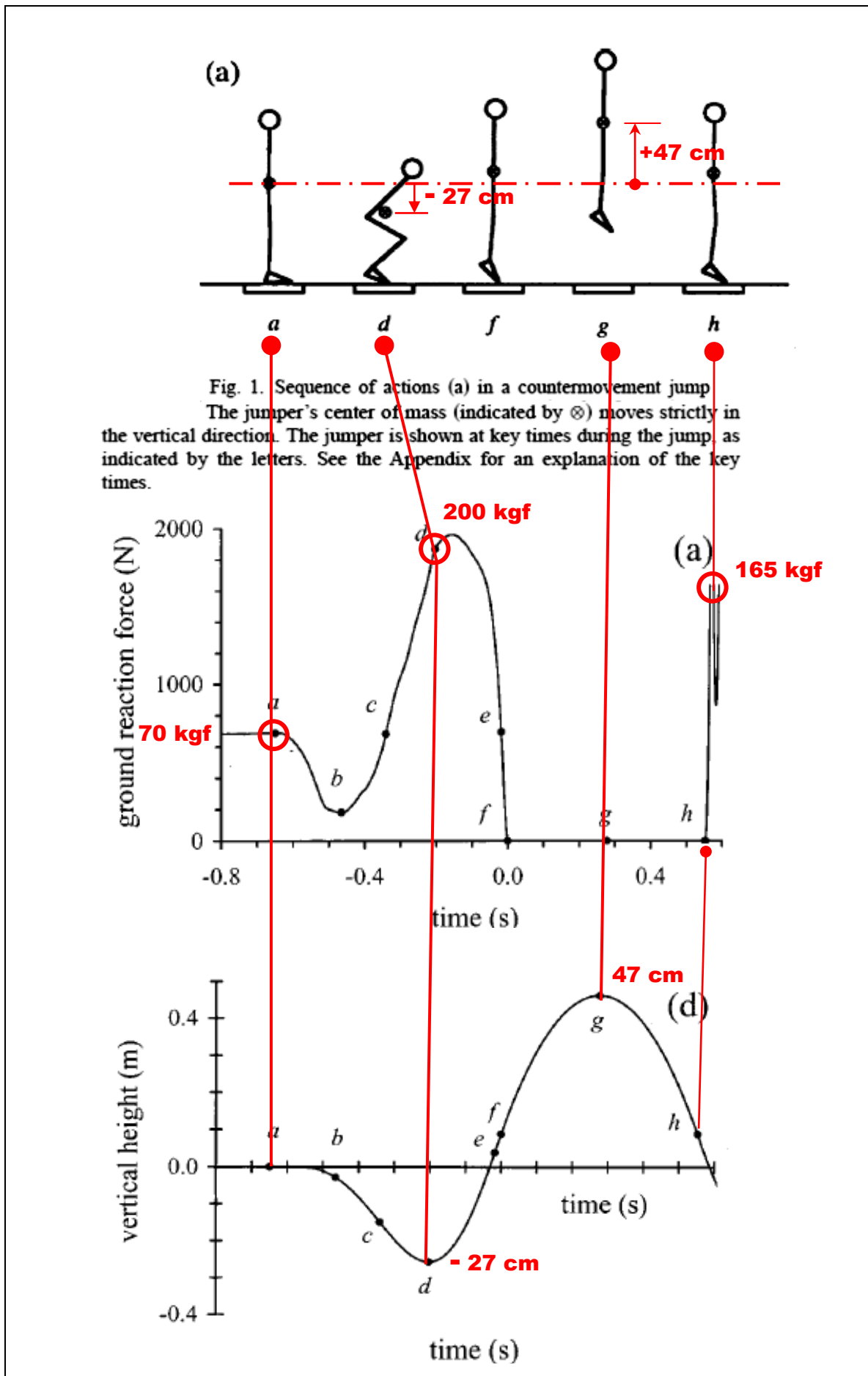
FORÇAS NO TRAMPOLIM

Analysis of standing vertical jumps using a force platform

Nicholas P. Linthorne - May 2001

School of Exercise and Sport Science. The University of Sidney, Australia

http://ebm.ufabc.edu.br/wp-content/uploads/2012/05/VerticalJump_Linthorne.pdf



Força máxima no piso = 2,85 x Peso do Saltador

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“ ESTUDO DO SALTO VERTICAL :Uma análise da relação de forças aplicadas”

EMERSON MIGUEL DA CRUZ - Dissertação de Mestrado

UNIVERSIDADE ESTADUAL DE CAMPINAS -- FACULDADE DE EDUCAÇÃO FÍSICA - 2003

http://repositorio.unicamp.br/bitstream/REPOSIP/275450/1/Cruz_EmersonMiguelda_M.pdf

1.4.2 - Fases do salto

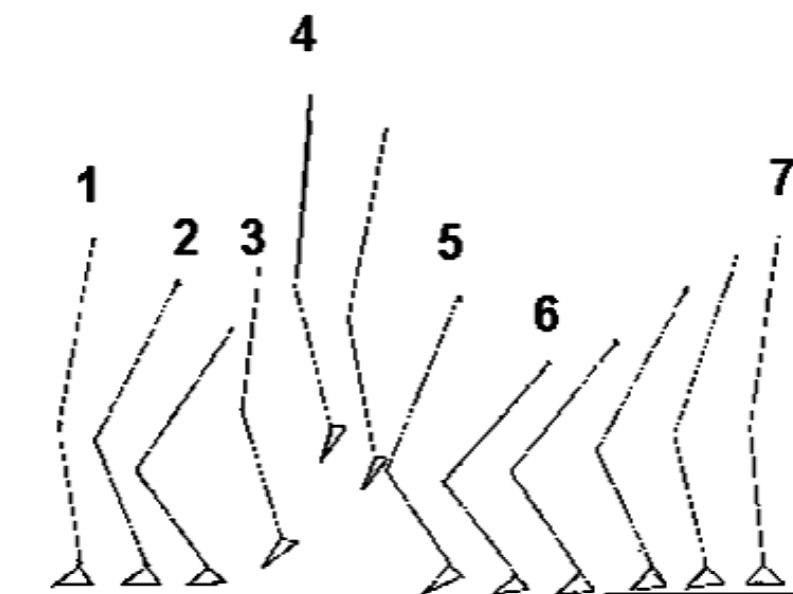


Figura 07: Diagrama de linhas representando as posições do membro para cima e para baixo durante as fases do salto. As posições do membro foram distribuídas da esquerda para a direita, e o intervalo de tempo entre cada diagrama é de 0,2 s (figura adaptada de Durward, Baer e Rowe 2001).

2.2.3 – Forças de reação do solo.

Para tal intento foi utilizado o Laboratório de Movimento da **ESSEFEGO** (Escola Superior de Educação Física e Fisioterapia de Goiás) que faz parte da **UEG** (Universidade do Estado de Goiás); as medidas da força de reação do solo foram registradas durante a execução de um salto vertical tipo Counter Moviment Jump (salto com contra movimento) obtidas através de um sistema de plataforma de força ligada a um computador, com seis câmeras de infravermelho sincronizadas e um sistema operacional próprio (Sistema Peak Mottus 6.0) para registro e apresentação dos valores e geração de um modelo virtual do salto executado.

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10 metros

7,5 metros

5 metros

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