Estruturas Projetadas, Construídas e Existentes

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1. DIFERENÇA ENTRE ESTRUTURAS PROJETADAS, CONSTRUÍDAS (OU EM CONSTRUÇÃO) E EXISTENTES

A. Estrutura Projetada

- Erro grosseiro está fora. Supõe-se os engenheiros preparados para evita-lo.

- Os γ aplicados aos valores característicos cobrem as Variabilidades Normais, limitadas pelas Tolerâncias definidas em Normas e Especificações relativas a:

- Ações – Permanentes e sobretudo Variáveis.
- Materiais – entregues na obra.
- Aproximações de projeto – para solicitações e resistências.
- Desvios de obra – geometria (seções, posição das armaduras, prumo, vãos, etc), homogeneidade e qualidade final dos materiais como aplicados.

B. Estrutura Existente – perto do fim da vida útil por exemplo

- Se aparecer um erro grosseiro, não há o que discutir, uma correção é imperiosa! – é o caso de problema estrutural claro ou sério de durabilidade – reforçar ou demolir!

- Se a estrutura tem muitos anos de bons serviços, isto é, estruturalmente em bom estado, precisando apenas de recuperação dos requisitos de durabilidade – duas medidas são necessárias: verificação especial e recuperação!

- Essa verificação especial pode ser feita com redução dos γ baseada nesses bons serviços e em levantamento de campo comprobatório de que os desvios de construção foram menores que as tolerâncias, legitimando o uso de γ menores.

Exs – 4 Obras do DER – 3 na SP304, 1 na SP147; Torre Itália

C. Estrutura Construída ou em construção

- Erro grosseiro está fora. Se identificado deve ser corrigido!

- Os desvios de construção devem respeitar todas as Tolerâncias definidas em Normas e Especificações relativas a:

- Ações – Permanentes e sobretudo Variáveis de execução
- Materiais – entregues na obra
- Desvios de obra – geometria (seções, posição das armaduras, prumo, vãos, etc), homogeneidade e qualidade final dos materiais como aplicados!

- Está previsto que a Operação deverá respeitar as Tolerâncias operacionais e executar a manutenção como previsto.

- Em princípio não há redução a fazer nos γ! a menos que se façam medidas em campo como por ex retirar testemunho do concreto quando o γ_c pode ser reduzido.

2. SAFETY OF EXISTING BRIDGES

IABMAS - 2016 F. Iguaçu Brazil

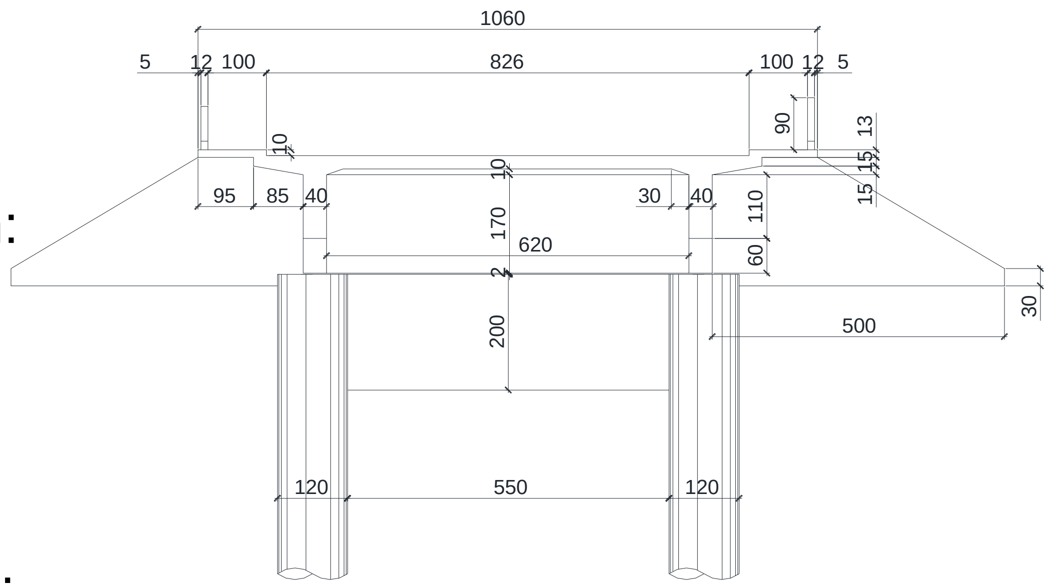
Following the New European Practice (Mancini 2010 e Vrouwenvelder 2010), these arguments justify the reduction of the partial safety factors, for the verification of a “well Built” structure, but it is important:

- To recuperate the durability defects of the structure;
- To follow the remaining service life, differently.

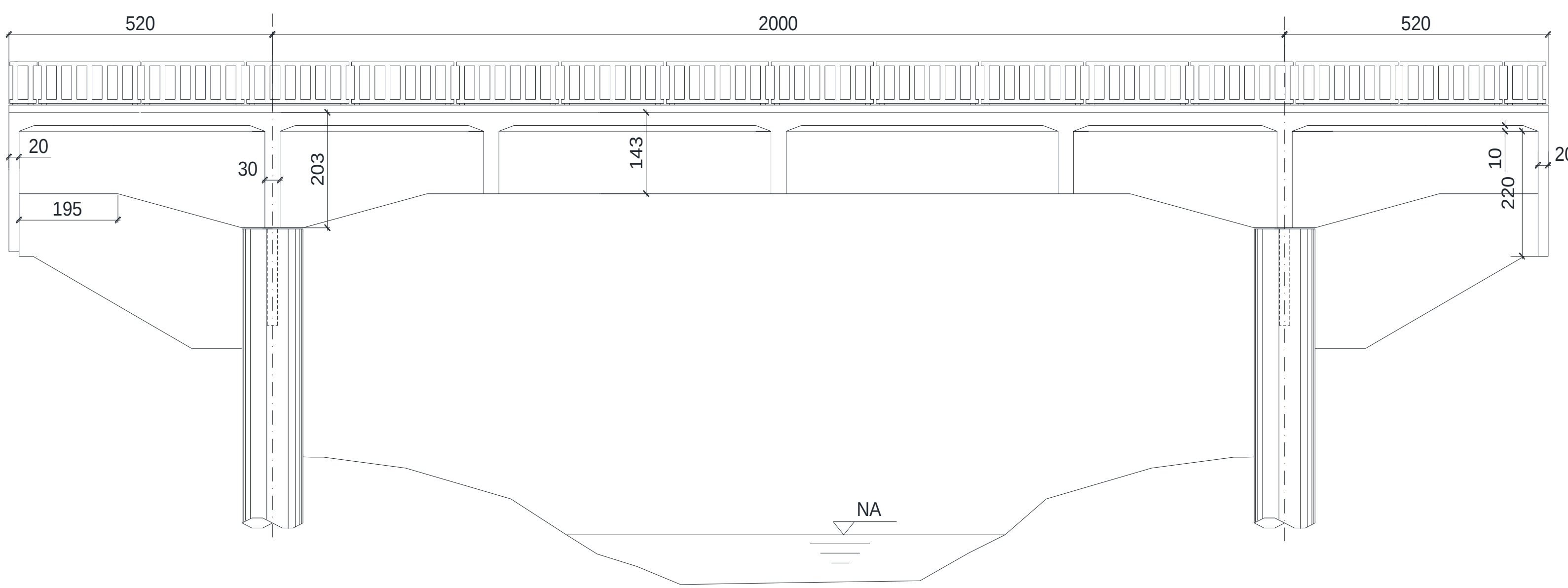
It is important to emphasize that, this reduction of design values of actions and resistances and even the durability requirements, through reduction of partial safety factors, do not increase the failure probability (Reliability Theory) required by the codes in the remaining service life

EXISTING BRIDGE – more than 50 years

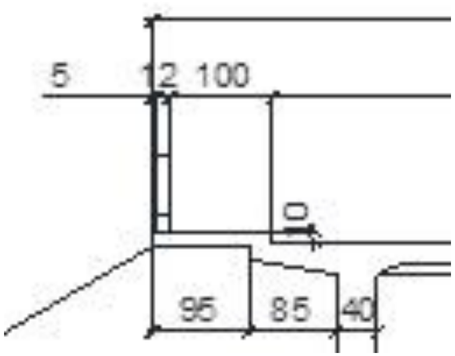
Transversal section:

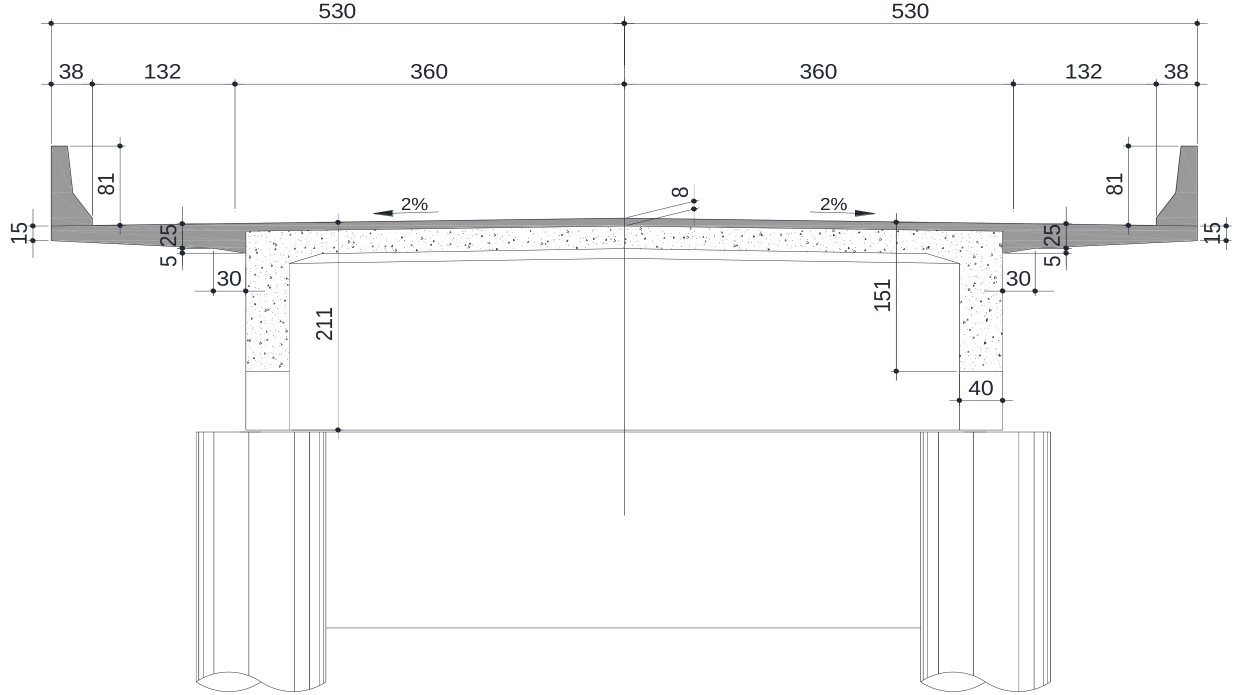


Longitudinal section:



REHABILITATION





- Rehabilitation consists of elimination of sidewalks, parapets and inclusion of New Jerseys at the extremity of transversal cantilever, increasing the total useful width of the bridge;
- Inclusion of concrete pavement over the slab;
- Execution in 2 phases, not to interrupt traffic.

DIMENSIONING

Parameters used for calculation of original e rehabilitated conditions were:

<u>Original</u>:

Concrete Code NB-1/1950
Highway load TB24/1950
Concrete Strength $\sigma_{c,28}$ = 20 MPa
Steel 37CA (yield stress= 320 MPa)
Total external safety factors:
F_g = 1,65
F_q = 1,20 x 1,65 ~ 2,00

<u>Rehabilitated</u>:

Concrete Code NBR6118/2003
Highway load TB45/1984
f_{ck} = 15 $\longrightarrow$ 30 MPa (I. Hammer)
f_{ck} = 25 MPa (slab complementary cap)
Steel 37CA (yield stress = 320 MPa)
Steel CA50 (yield stress = 500 MPa – Cantilever strengthening)
Partial safety factors:
γ_c = 1,4 γ_s = 1,15
γ_q = 1,35 γ_q = 1,50

DIMENSIONING

Internal forces – Original Bridge – Original Code:
(only external safety factors)

Section	M_g (tf.m)	1.2*M_q (tf.m)	M_d (tf.m)	A_{so} (cm²)	x/d
Span	211,6	357,4	938.9	252,5	0,17
Support	-161,6	-215,5	-622,2	121,0	0,33

Internal forces – Rehabilitated Bridge – Present Code:
(partial safety factors for actions and resistance)

Section	M_g (tf.m)	M_q (tf.m)	M_d (tf.m)	A_{sr} (cm²)	x/d	A_{sr}/A_{so}
Span	268,8	384,9	940,2	310,7	0,25	1,23
Support	-182,4	-241,3	-608,2	177,7	0,80	1,47

REDUCTION OF SAFETY MARGIN

Following Reliability Theory, after 50 years of good services and local verification of strength and geometry, the bridge was considered “Well Built”, and action partial safety factors could be reduced:

$F_{d,original} = 1{,}65 \times (F_g + 1{,}2F_q)$

$F_{d,original,present} = 1{,}35 \times F_g + 1{,}5 \times F_q$ (Design Forces on original bridge by NBR 6118/2003)

$F_{d,rehabilitated} = 1{,}35 \times F_g + 1{,}5 \times F_q$

$\boxed{\bar{F}_{d,rehabilitated,reduced.} = 1{,}20 \times F_g + 1{,}35 \times F_q}$ - following bibliography.

$\Delta 1 = F_{d,rehabilitated} \,/\, F_{d,original,present}$

$\Delta 2 = F_{d,rehabilitated,reduced} \,/\, F_{d,original,present}$

REDUCTION OF SAFETY MARGIN

- Comparing flexural moments:

Section	$M_{d,orig}$ (tf.m)	$M_{d,orig.pres.}$ (tf.m)	$M_{d,rehab}$ (tf.m)	Δ1
Span	938,9	732,4	940,2	1,28
Support	-622,2	-487,5	-608,2	1,25

Section	$\overline{M}_{d,rehab,red}$ (tf.m)	Δ2
Span	842,2	1,15
Support	-544,6	1,12

- Even with the reduction of internal forces, due to its “Well Built” quality, defined by geometry and materials qualities, there was an increase of forces in relation to original design, bough analyzed with the present code. This way it would be necessary to strengthen the beam for the reduced forces.
- It was decided to increase the negative steel as we will strengthen the transversal cantilever.
- The increase of the positive steel would be more difficult if we use standard steel.

RELIABILITY THEORY

Case	A	B	C	D
Beta	3,1	3.3	3.3	3.5
Fail Prob.	1E-3	5E-4	5E-4	3E-4

Reducing needs by reducing variabilities:

A – Positive Moment – design condition, f_{ck} 15 MPa and CV 15%

B – Positive Moment – measured f_{ck} 30 MPa and CV10%

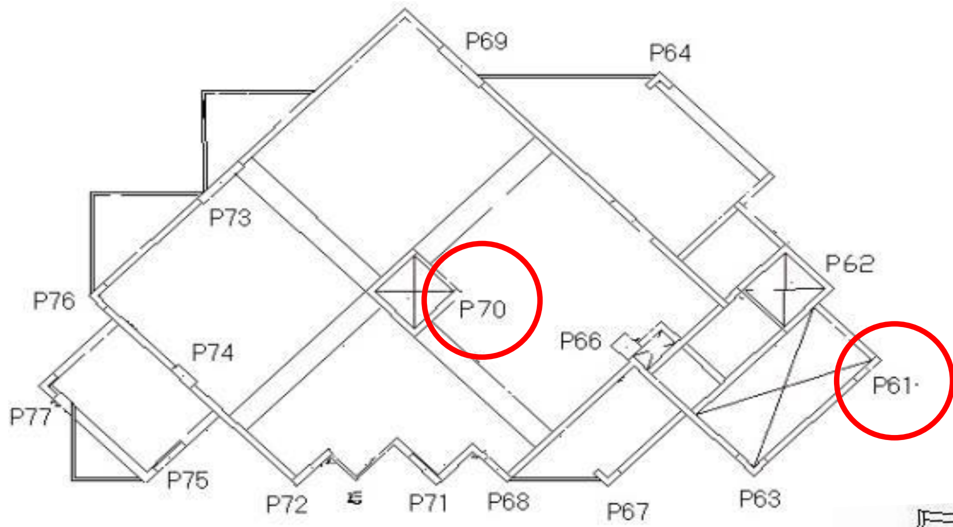
C – Positive Moment – f_{ck} 15 Mpa and reduction of CV (h) from 6 para 3%

D – Positive Moment – B with CV (f_{ck}) 15% and CV (h) 3%

As the failure probability of cases D is acceptable by the indicated bibliography, and also by the Brazilian code, no strengthening was made at mid-span. **Decision approved by the DERSP in 2012.**

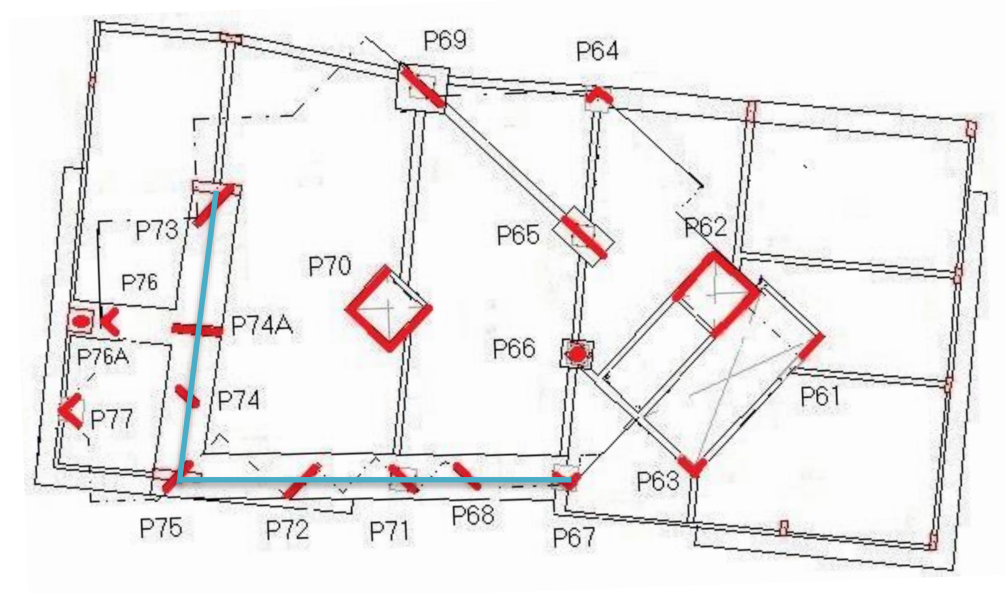
3. ITALY TOWER COLLAPSE

3.1 Data - 16 floor tower and attic built above a common use floor (PUC)



Typical floor

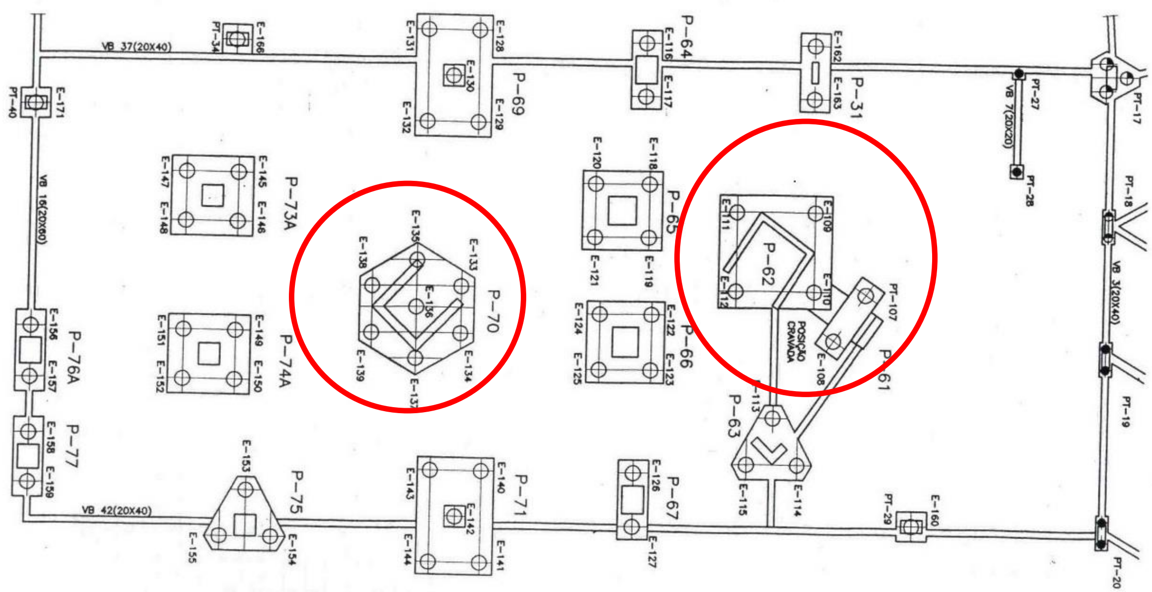
PUC floor



ITALY TOWER COLLAPSE

- Below the ground floor are the foundations (no basements) built up of caps on Franki piles ϕ52 cm.
  These piles were performed with lengths around 10 m.
- The profile of the subsoil consists of a thick layer of loose clayey sand, based on a compact sand layer
- on top of the sandstone. The piles were practically supported on sandstone. The water level is 3 m deep.

Foundation Structure



ITALY TOWER COLLAPSE

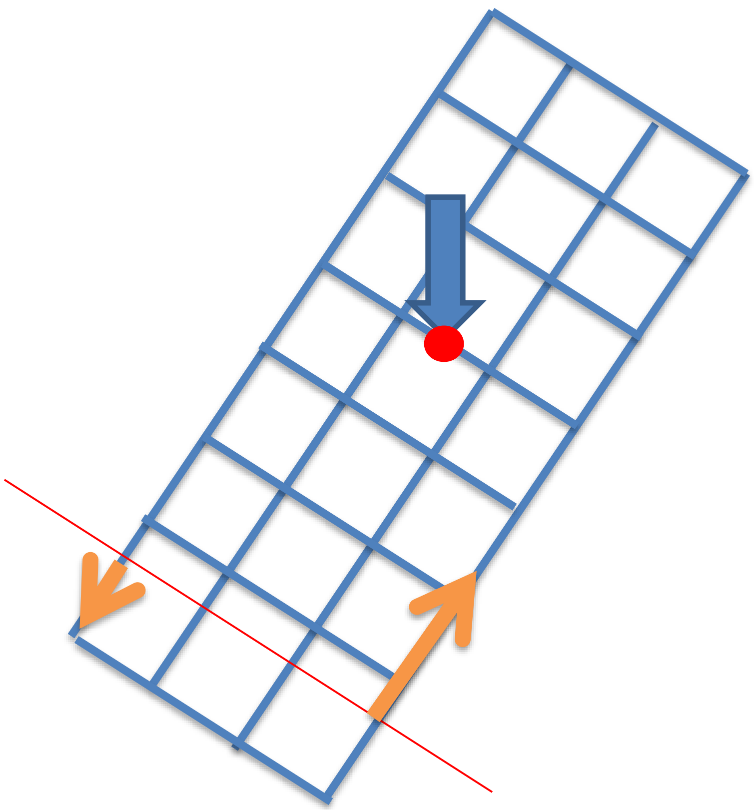
3.2 Fotos - **17-10-1997**

Inclination of P70 after collapse

Tensile failure at PUC level

Floors upside down

Column axial forces
just before collapse



3.3 COLLAPSE description

a) Before the collapse, the building showed no signs of anomalies. Age – one year

b) Collapse began with a bang at 2:00 in the morning.

c) The glass and the facade frame broke due to significant differential settlements.

d) At 6:00am Italy Tower would have rotated a bit around his vertical axe, opening the joint with Spain Tower and started settle slowly as a whole much more in the back than in the front and rotate till an inclination of ~ **1:1!!??** Suddenly and quickly the structure collapsed.

3.4. Safety verification - **1998**

A.Structural safety

- Designed Structure

$Sd = \gamma_{fg1} S_{g1} + \gamma_{fg2} S_{g2} + \gamma_{fq} (S_q + 0, 8_w)$ with $\gamma_{fg1} = \gamma_{fg2} = \gamma_{fq} = 1.4$

Resistances were: concrete f_{ck} = 18 MPa, $\gamma_c = 1.4$ and steel f_y 500MPa, $\gamma_S = 1.15$

- Structure as Built

Due to good field measurements (geometry and concrete strength) new safety factors were: $\gamma_{fg1} = 1.3$, $\gamma_{fg2} = 1.25$, $\gamma_c = 1.26$.

- Collapse Hypotheses

Collapse hypotheses - every combination imagined with:

$\gamma_{fg1} = \gamma_{fg2} = \gamma_{fq} = 1.1$; $\gamma_c = 1.1$ and $\gamma_s = 1.0$

B.Foundation safety

We adopted the verification of the piles by working loads, as usually, what brings us to accept the limits; 1500kN for vertical loads and 1500 x 1,3 = 1950kN when wind forces are considered. The ultimate structural capacity was 2400kN for concrete of f_{ck} 14 MPa

3.5. Verification Results

- Designed structure

- No problems were detected in the foundations.
- Elastic model shows that the stability parameter γ_z reaches acceptable 1,13, but some members forces were higher than resistances and we should consider them cracked.
- Cracked model shows that the stability parameter γ_z increased to still acceptable 1,24 and only a small number of members forces were a little higher than resistances.

- Structure as built

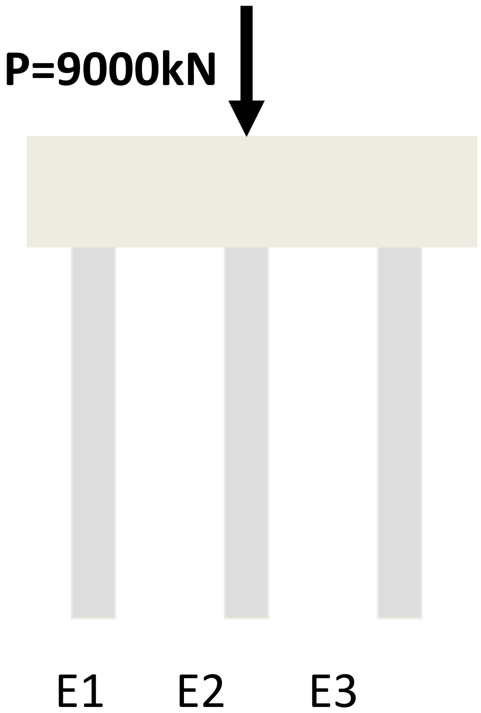
- Due to load increasing, load foundation were not acceptable: considering only vertical actions, the pile load were critical in the P61/62 ranging from 1440 to 1770kN. For P70 pile loads vary from 1740 to 1790kN. Values higher than 1500kN and not acceptable.
- For the structure itself, the increase of concrete strength compensates the increase of loads

- Collapse Hypotheses

 - All collapse pictures include additional construction errors.
 - Eccentricity of the columns born from the transition beam, Pile defect, Pile positioning errors. These Hypotheses do not identify a brittle failure able to explain the collapse.
 - Piles with different stiffness supporting the same cap. A research on ϕ52 Franki pile tests, concentrated in short ones (from 6 to 10m long) with point in rock, shows that their stiffness could change a lot, between 1 to 3,5.

3.6. Piles with different stiffness supporting the same cap.

Hypothetic Example



Case 1: Uniform piles -- R1=R2=R3

N1=N2=N3=3000kN

Case 2: Non uniform Piles -- R1'=R3'=R2'/3

N1'=N3'=900/5=1800kN

N2'=3*180=5400kN

N2'=1.8*N2

ITALY TOWER COLLAPSE

The results show in first place a great sensitivity to the variation of stiffness. Secondly they show that it is not difficult to obtain pile loads overpassing the established structural limit between 2400 and 2800kN.

The maximum pile load found were 3570kN, applied to the pile E108.

3.7 - CONCLUSIONS

We consider, therefore, **that the decisive factor of the collapse was the application of the conventional criterion of uniformly stiff piles to a case of short piles with point in rock were the geotechnical load capacity is much higher than the structural strength.**

As we found no other cause for the collapse, we decided to define this one as the probable cause, and suggest a change in our related codes.