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On the Problem of Temporary Structures

Considérations sur les constructions provisoires

Über provisorische Tragwerke

Wolfgang J. OBERNDORFER
Professor
Technical University
Vienna, Austria

Wolfgang J. Oberndorfer, born 1941, received his diploma at the Technical University of Vienna and a Master's degree at the University of California, Berkeley. After 16 years of service in construction companies he was appointed professor for construction economics and planning techniques.

SUMMARY

The special features and the safety of temporary structures as well as the respective Austrian Codes are discussed. The reason for the collapse of a scaffolding are shown.

RÉSUMÉ

Les caractéristiques et la sécurité de constructions provisoires et les normes autrichiennes correspondantes sont discutées. Les raisons de l'écroulement d'un échafaudage sont données.

ZUSAMMENFASSUNG

Die Eigenschaften und Sicherheit von temporären Tragwerken und die entsprechenden österreichischen Vorschriften werden besprochen. Die Ursachen des Einsturzes eines Fassadengerüstes werden gezeigt.

1. SPECIAL FEATURES OF TEMPORARY STRUCTURES

We can define these features with respect to permanent structures:

- Shorttime use for temporary purpose
- Frequently not intended to be used by the public; therefore loads are better known. Improved control during peak loads.
- Choice of material and framework according to purpose and feasibility not according to aestethics.
- Parts are intended to be reused as parts of a modular system.
- Need to verify strength of used material.
- Frequently the execution of temporary structures is not as accurate as of permanent structures, therefore control by the engineer himself is mandatory. The engineer can improvise more than with permanent structures.
- Possibility to save costs due to the fact that codes need not to be observed as strictly as in construction of permanent structures. By taking more responsibility the engineer or the construction company can save money. However this requires better understanding of the behaviour of structures than standard calculations.

For the design of temporary structures we set up three special rules:

- The engineer must be aware how accurately the model represents the real structure.
- The engineer needs to calculate only as accurately as it is required and not more accurately than the model represents the real structure. Otherwise he would pretend unrealistic accuracy.
- The engineer must take into account possible failure due to loss of stability. 85 % of all failures in temporary structures can be traced to this cause.

In Austria, calculation and design of temporary structures is up to the construction company and is done by an employee of the company or by a consulting engineer ordered by the company. The design and calculation of falsework and formwork of bridges has to be submitted to the client.
It has been proved many times that failures of structures are the result of the superposition of design and construction flaws. Usually, no standard structure fails, the American Institute of Architects discovered in Spring 1981 in a broad investigation of spectacular failures during the last three years. In this report, the Institute made the following conclusions:

- Failures are never caused by one reason only.
- Failures occur usually in those main structural elements which have no possibility of load redistribution.
- Loss of stability is usually a reason.
- Influence of secondary stresses is often a reason.
- If the engineer does not have the right feeling for the behaviour of the structure, based on experience, simple confidence in codes or computer programs may not be good enough.

For temporary structures the 2nd, 3rd and 5th conclusion are of main importance. We add a 6th conclusion:

- lack of fine work and lack of control provoke failures.

2. ON THE JUDGEMENT OF SAFETY

Let us repeat some aspects of the philosophy of safety of structures, so we have a better judgement for failures of structures: An absolute safety against failure of structures is usually not possible. The safety coefficient is defined as the ratio of the probable load bearing restistance R_k to the probable maximum loading stress S_n. (Fig. 1).

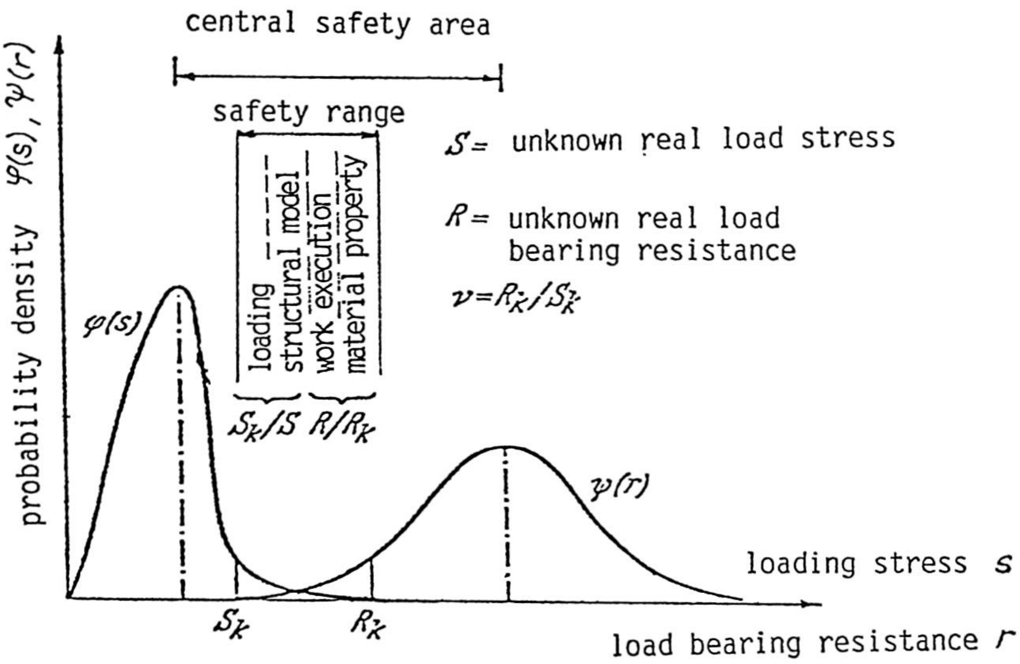

Fig.1: definition of safety coefficient

Real loading stress S and real load bearing resistance R are not known exactly but statistically distributed more or less well known with φ (s) and ψ (r). We define the value of material strength by demanding that only 5 % of all tests of material strength lead to failure of the material. This implies that the area below the Gaussian curve will be divided 5 : 95 by R_k. Similarly we depict the load stress and then we define

$$\nu = R_k / S_k.$$

This safety margin is furthermore depending on:

- choice of structural model, added up with the load usually,
- accuracy of construction, added up with the material strength usually.

If we define with ν_i the safety coefficient for the influence i (ν_i >1) then we can write generally:

$$\nu = R_k / S_k = \prod_{i=1}^{n} \nu_i$$

The magnitude of ν for temporary structures should not simply be chosen from the code because our special circumstances of our special temporary structure are usually not covered in the code and we may choose a safety coefficient which is too small or unnecessarily large. We should take into account in addition:

- probability distribution of load and material strength

- risk when structure collapses
- state of structure that has to be prevented by the safety margin: state of non-serviceability of state of collapse or both of them.

3. CODES CONCERNING TEMPORARY STRUCTURES (TS) IN AUSTRIA

3.1 Survey

In Austria there are three types of codes for temporary structures:
(1) Codes for design and calculation of TS, numbered B 4....
(2) Codes for construction of TS, numbered B 2....
Both of them can be agreed upon by client and construction company or not.
(3) The federal decree protecting employees and workers (DSV) representing compulsory law.
By surveying these codes and the DSV we found:

type of TS	Code B	DSV
timber structures	4102/II	-
scaffoldings	4007	x
falsework for R.C.	4200/II	x
support to trenches	2503	x
steel supports for excavation	2205	x
anchoring in tunneling	4455	x
supports in tunneling	2203	x

The "x" in column "DSV" indicates that the DSV also covers this area (more or less extensively). Furthermore we found the strange facts, that the DSV

- partly repeats the codes,
- partly contents more precise instructions,
- partly contents different instructions.

3.2 Instructions concerning TS

Steel- and RC-structures: no particular instructions.

Timber structures (B 4102/II): no exemptions for use of timber in TS, contrary to the observation in (1). Stength has to be reduced about 15 % for timber piles for bridges.

Scaffoldings (B 4007): the general loading and design instructions have to be applied. Standard scaffoldings are allowed without particular calculation. Deflection of timber: twice as much allowed as for permanent structures. Precise instructions on timber coverings, spans, exceeds of coverings, bulwarks.
The DSV repeats the instructions; partly there are stricter instructions.

Falsework (B 4200/II): the general loading and design instructions have to be applied. For struts there are particular instructions(i.e. number and situation of joints). For struts there exist minimum dimension ($\phi \geq 7$ cm) and bracing instructions (when span is ≥ 4 m).
The DSV repeats the code.

Supports to trenches of sewers (B 2503): the code gives some design instructions (p.e. supports 10 cm above surrounding level, thickness of timber post ≥ 5 cm, distance between supports ≤ 3 m, exceeds ≤ 50 cm).
The DSV complements the code by requiring supports to trenches below 1,25 m depth.

Steel supports for excavation (B 2205): the safety coefficients may be lowered about 10% if the steel supports stay no longer than six months.

Anchoring in tunneling (B 4405): corrosion protection may be lowered for temporary anchors (< 2 years).

Supports in tunneling (B 2203): the general instructions apply for the design of supports regardless whether the supports are temporary or permanent.

4. EXAMPLE

The building of the Ministry of Education and Culture in Vienna (Bankgasse) was given a new facade, approx. 55 m (width) x 21 m (height). The building has four floors and was made of brick in 17th and 18th century. The construction company used steel scaffoldings that collapsed three weeks after assembling totally. Since there were no people on and below the scaffoldings accidentally nobody was injured. We had to find out the reason for the collapse.

(1) Check of number of anchors.

Counting the holes for dowels we found 46. The code requires 1 anchor for 25 m^2, that leads to 45 (okey).

(2) Structural analysis (sketch of scaffoldings s. Fig. 2).

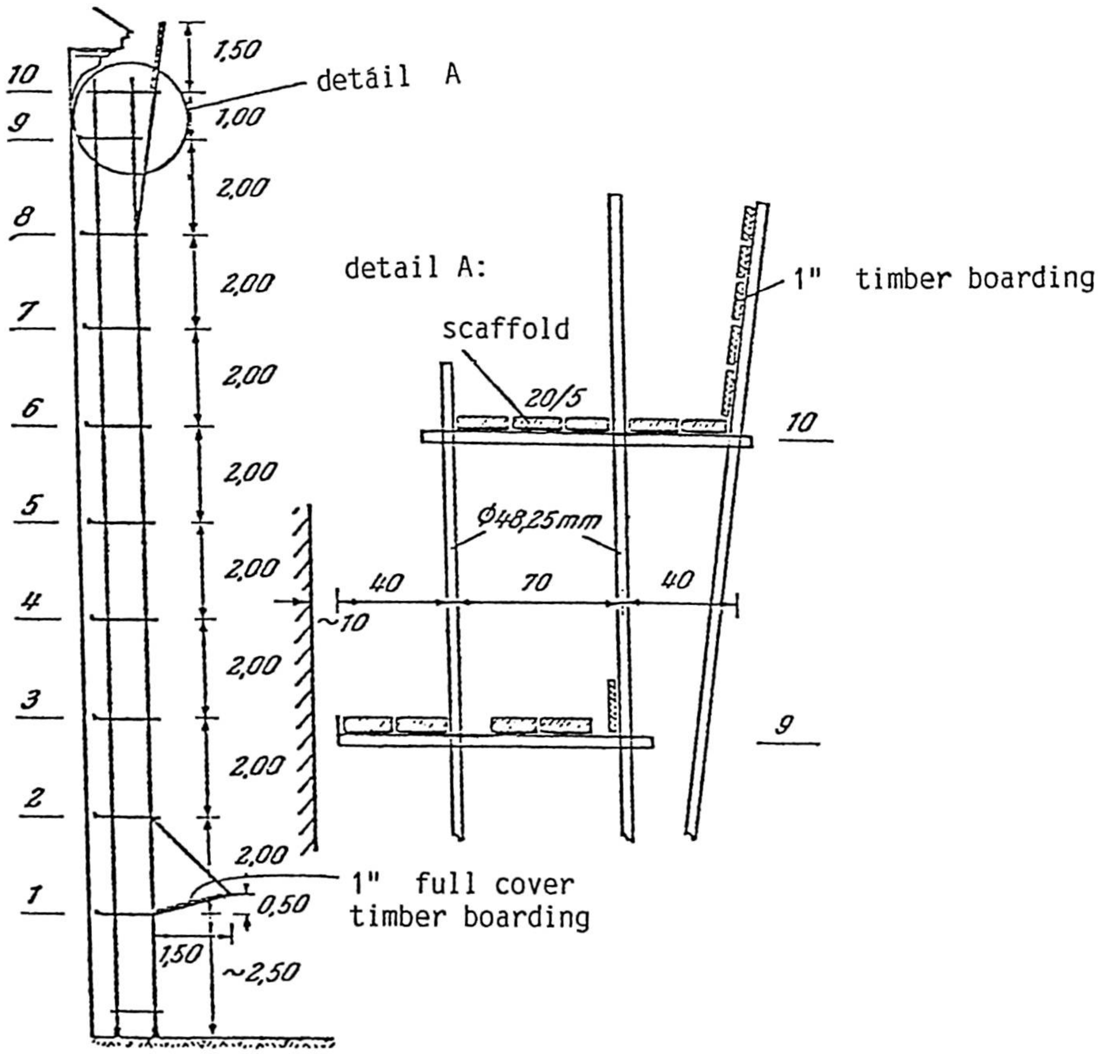

Fig.2: cross section through scaffoldings with detail

The loading assumptions were:

dead load 0,40 KN/m
live load 1,20 KN/m
wind load 0,38 KN/m² - 50% (net instead of cloth)

The engineer found an existing extraction force of

H exist = 3,7 KN.

Making the assumptions

compression stress in brick wall 0,80 KN/cm²
friction coefficient 0,65

the engineer found a permissible extraction force of 4,9 KN.
This gives a safety coefficient of

$$\nu = \frac{4,9}{3,7} = 1,22$$

Before the collapse, ν was even expected to be higher because the scaffoldings were not loaded (ν = 4,9/2,8 = 1,75)! Unfortunately the structure did not behave that way.

(3) Investigation of further possible reasons for the collapse.

Bumping by cars on the struts had to be eliminated due to witnesses. Touching by the crane jib had equally to be eliminated after consulting the crane operator and due to witnesses. So our suspicion centered on:

- extraction resistance of the dowel,
- wind loading (magnitude of wind force at time of accident, lessening coefficient net instead of cloth).

(4) Dowel tests.

We made seven extraction tests on the facade of the building. Fig. 3 shows the dowel.

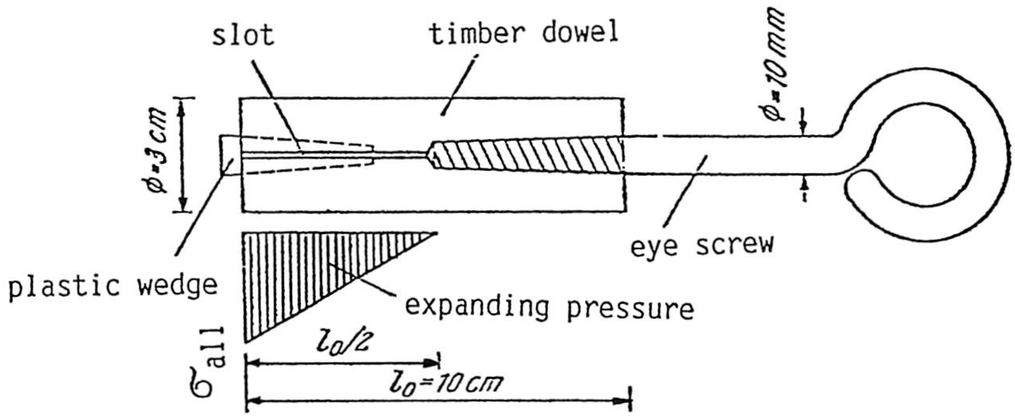

Fig.3: timber dowel and
transfer of extraction force

The results were as follows:

Test item	Extraction force (KN)	Reason of collapse	Depth of dowel hole (cm)	Age of brick wall (years)
1	1,20	dowel sliding	6,5	300
2	4,28	extraction of eye	6,5	300
3	2,97	dowel shearing off	9,0	150
4	1,26	dowel sliding	10,0	300
5	4,68	opening of eye	10,0	150
6	1,73	dowel shearing off	10,0	300
7	4,70	opening of eye	10,0	150

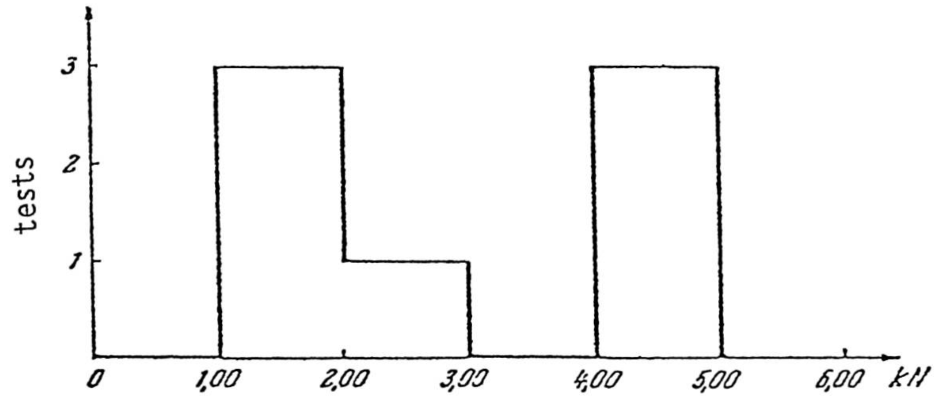

Fig.4: histogram of dowel tests

Fig. 4 represents the histogram of the results.

A significant influence of the depth of the hole was not found. The transfer of the extraction force occurs along the rear half of the dowel which is usually well embedded in the brick. The circumferential compression stress decreases as shown in Fig. 3 and must not be assumed as constant along the entire depth as it was done in the structural analysis.

(5) Wind tests

A study of the literature dealing with the windforce on nets did not yield sufficient information [see (5) to (9)]. Results for wire nets and sieves could only indicate the upper and lower boundaries for the wind force. Therefore wind channel tests were carried through. The maximum wind force coefficient was found to be 0,6 for a net with a fullness coefficient of φ = 0,45. The table below shows the wind force coefficients drawn from literature and indicates the difference between the correct and an auxiliary value.

Literature	structural member	wind force coefficient	(%)
Zuransky (5)	antenna net	0,80	+ 33
Rosemeier (6)	truss	1,20	+ 100
EMPA(7) + B 4014(4)	net with fullness coefficient of φ = 0,53	0,95	+ 58
Sachs (8)	Wire nets and sheet metal with holes	0,67	+ 12
Idel'chick (9)	sieves	0,55	- 8
B 4014 (4): 5.2.3/1	tightly stretched cloth	1.80	+ 200
5.2.3/2	flag	0,46	- 23
5.4.1/	truss	0,63÷0,96	+5 ÷ 60

In the structural analysis 50% of the value 0,46 was taken because the net was assumed to be a permeable flag.

(6) Final remark

The reasons for the collapse of the scaffoldings were found to be:
- underestimating the wind forces on nets,
- overestimating the extraction force of dowels in brick walls.

It is a sad fact that the structural engineer and the site engineer acted in compliance with the concurrent codes. This accident shows the necessity for the engineer not only to read the codes critically but also to understand the path of the forces from their point of appliance into the foundations and/or anchorages. The education of our engineers at the universities has to take into account this requirement.

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