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# Check of Roof Loading of the Westfalenhalle in Dortmund

## Détermination des charges du toit de la Westfalenhalle à Dortmund

## Kontrolle der Dachlasten bei der Westfalenhalle in Dortmund

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**SUMMARY**

In the course of strengthening the roof of the Westfalenhalle, it was necessary to check the actual roof loading. This article shows how the loads were indirectly determined by measuring the anchor forces. These were obtained by means of oscillation measurements and an exact interpretation of the frequency spectrum.

**RÉSUMÉ**

Au cours des travaux de renforcement du toit de la Westfalenhalle les charges effectives du toit ont dû être vérifiées. Cet article décrit la méthode selon laquelle les charges sont déterminées indirectement en mesurant les effets dynamiques sur les tirants. Ces influences sont déterminées en mesurant les oscillations et à l'aide d'une analyse du spectre de fréquences.

**ZUSAMMENFASSUNG**

Im Zuge der Verstärkungsarbeiten an der Dachkonstruktion der Westfalenhalle war es erforderlich, die tatsächlichen Dachlasten zu überprüfen. In diesem Beitrag wird gezeigt, wie diese Lasten indirekt durch die Bestimmung der Zugstangenkräfte festgestellt wurden. Diese Kräfte wurden durch Schwingungsmessungen und eine genaue Analyse der Frequenzspektren ermittelt.

## 1. DESCRIPTION OF THE HALL

The 40 year old Westfalenhalle in Dortmund has a spectator capacity of approx. 12000 - 14000. The plan dimensions of the building are approx. 110 m x 98 m in a semi-elliptic shape, with a clear height of approx. 28m.

The main structural roof elements consist of 20 riveted steel beams, which cantilever out to the centre of the roof, and are anchored at the rear by means of tie rods from the end of the arm down to the foundations (Fig. 1).

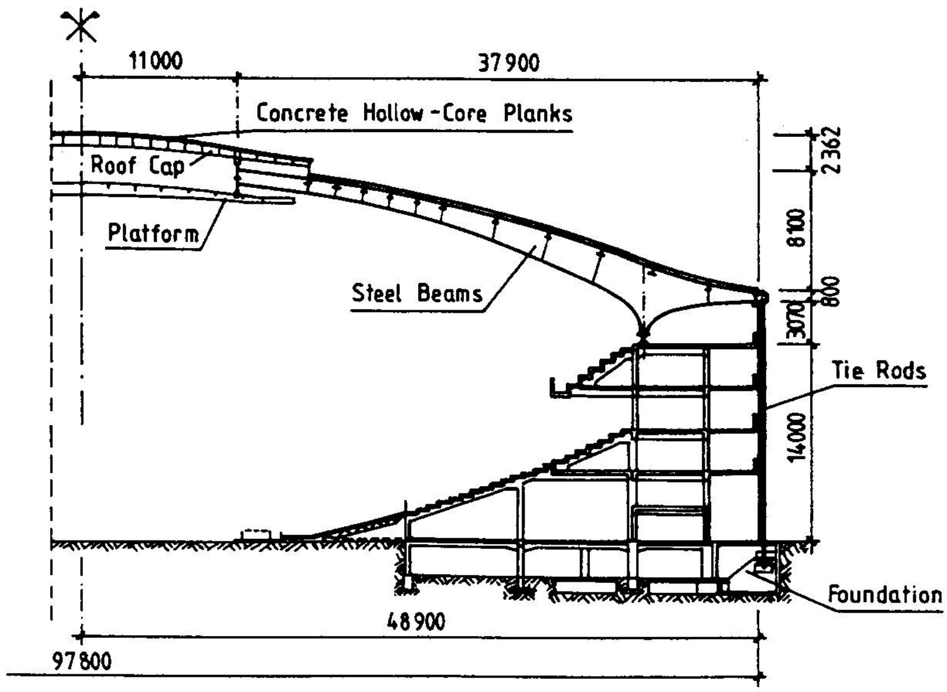


Fig. 1 Cross Section of the Westfalenhalle

The tie rods each consist of 5 separate bars with diameters varying from 55 mm at the weakest beam to 110 mm at the strongest. The clear length of the tie rods from foundation level to the beam connection is 17,34 m.

The high roof weight of approx. 1,30 kN/m$^2$ is due mainly to the concrete hollow-core planks used originally 40 years ago.

## 2. THE PROBLEMATIC NATURE OF THE ROOF LOADS

It became necessary to check the roof structure in 1992, due to the fact that beam loading during rock concerts was ever-increasing.

The total loading due to loudspeakers and stage lighting was often in the region of 300 kN. Also, due to various previous structural alterations to the roof, there was some uncertainty as to the present roof load. In order to have a safe basis for checking the structure, as well as for the preparation of any structural strengthening which might be required, the loads were indirectly checked by measuring the tie rod forces. These forces were obtained by measuring the natural frequencies of the tie rods.

## 3. TEST DATA AND MEASURING INSTRUMENTS

Seismic acceleration absorbers with a range of 0,01 Hz - 1200 Hz were connected to the tie rods.

The output signal from these acceleration absorbers was amplified and transferred to a monitor. In this case, the five first natural frequencies could be obtained and depicted with a great degree of accuracy.

By means of various test measurements, the influence of the natural frequencies of the absorber, the influence of mutual neighbouring rods and the influence of transverse vibrations were eliminated.

The typical simplified test result for a tie rod with a diameter of 100 mm is shown in Fig. 2. The distinct peaks of the natural frequencies 1 to 5 are clearly shown.

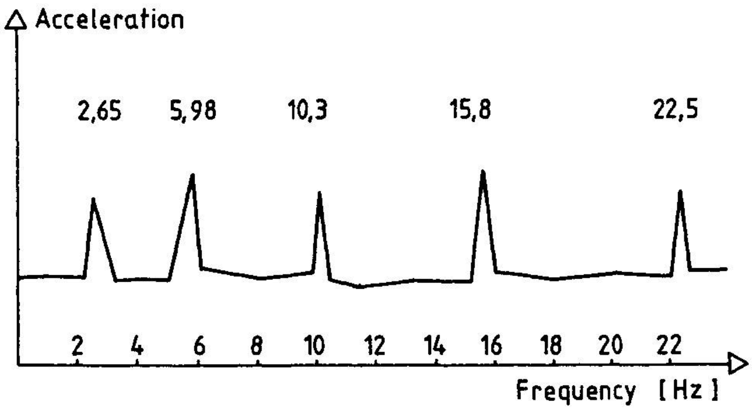


Fig. 2 Test Data (simplified) of a Tie Rod with 100 mm Diameter

First analysis of the test data led to an extremely disturbing result concerning tie rod forces, and thus roof loading. Flexible cable characteristics had been assumed for first analysis. Initially, this seemed to be appropriate for a frequency ratio of $f_2/f_1 = 2{,}26$, where $f_1$ = 1st natural frequency, $f_2$ = 2nd natural frequency. For a flexible cable this ratio is $f_2/f_1 = 2$ and for a simply supported bending member $f_2/f_1 = 4$.

However, using this simplifying first assumption, tie rod forces were obtained which were almost twice as large as expected.

The influence of the bending stiffness and of the support conditions on the actual forces is of great importance, and must be considered by further analysing the frequency spectrum, as shown in the following.

## 4. EVALUATION OF THE TEST DATA

### 4.1 Calculation of tensile forces

#### 4.1.1 Frequency spectrum of a flexible cable

The natural frequencies $f_n$ of a flexible cable under a tensile force can be obtained from the equation:

$$f_n = n \cdot \sqrt{\frac{Z}{0{,}408 \cdot g \cdot l^2}} \quad [\mathrm{Hz}] \tag{1}$$

where $g$ = weight of cable in kN/m
$l$ = length of cable in m
$Z$ = tensile force in kN
$n$ = number of natural frequency

Thus for the natural frequency No. n:

$$f_n = n \cdot f_1 \tag{2}$$

If, for the tie rod of 100 mm diameter, these values are combined to a frequency spectrum, where the first natural frequency is equal to the measured frequency, a spectrum according to Table 1, line 1 is obtained, which does not conform with the test data.

| | Support Condition | Frequency Spectrum Natural Frequencies No. 1 - Nr. 5 | Tensile Force |
|---|---|---|---|
| 1 | Z EI = 0 Z Free Cable | 2,65 5,3 7,95 10,6 13,25 Hz | Z = 521 kN |
| 2 | Z EI Z | 2,65 5,79 9,80 14,90 21,20 Hz | Z = 487 kN |
| 3 | Z EI Z | 2,65 5,88 10,08 15,42 22,08 Hz | Z = 393 kN |
| 4 | Z EI Z | 2,65 5,97 10,36 15,96 22,84 Hz | Z = 290 kN |
| 5 | Test Data | 2,65 5,98 10,3 15,8 22,5 Hz | Z ≈ 338 kN |

Table 1 Spectrum of Natural Frequencies for various Support Conditions

The tensile force Z of a flexible cable is obtained from:

$$Z_{cable} = 0{,}408 \cdot g \cdot l^2 \cdot \frac{f_n^2}{n^2} \tag{3}$$

Using this equation, different tensile forces for each natural frequency (see Table No. 2) are obtained from the test data of Fig. 2.

| | | 1 | 2 | 3 | 4 |
|---|---|---|---|---|---|
| | | Free Cable | Z Z | Z Z | Z Z |
| n | $f_n$ [HZ] | $Z_{cable,n}$ [kN] | $Z_{2,n}$ [kN] | $Z_{3,n}$ [kN] | $Z_{4,n}$ [kN] |
| 1 | 2,65 | 521 | 487 | 393 | 290 |
| 2 | 5,98 | 663 | 528 | 414 | 290 |
| 3 | 10,30 | 875 | 570 | 431 | 280 |
| 4 | 15,80 | 1158 | 617 | 449 | 268 |
| 5 | 22,50 | 1502 | 656 | 457 | 246 |

Table 2 Tensile forces Z [kN] for various Support Conditions

Considering these different values, it is obvious that the test data does not represent a flexible cable, because in reality there is only one actual tensile force in the tie rod. Therefore, it is essential to consider the bending stiffness and the support conditions.

### 4.1.2 Frequency spectrum and tensile forces with regard to bending stiffness and support conditions

The natural frequencies $f_n$ with regard to bending stiffness, support conditions and tensile force can be determined by using the following equation (see ref. 2):

$$f_n = n \sqrt{\frac{\frac{Z}{k_1} - k_2 \cdot N_E}{0{,}408 \cdot g \cdot l^2}} \tag{4}$$

where

$$N_E = \frac{\pi^2 EI}{l^2}$$

$k_1$, $k_2$ factors, related to $\frac{Z}{N_E}$ and the support conditions (see Table 3)

| | Z EI Z | | Z EI Z | | | | | Z EI Z | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | $k_1$ für $Z/N_E$ = | | | | | $k_1$ für $Z/N_E$ = | | | | |
| n | $k_1$ | $k_2$ | 10 | 100 | 1000 | ∞ | $k_2$ | 10 | 100 | 1000 | ∞ | $k_2$ |
| 1 | 1 | 1 | 0,897 | 0,949 | 0,979 | 1 | 2,441 | 0,836 | 0,908 | 0,959 | 1 | 5,140 |
| 2 | 1 | 4 | 0,928 | 0,956 | 0,980 | 1 | 6,406 | 0,873 | 0,918 | 0,961 | 1 | 9,760 |
| 3 | 1 | 9 | 0,947 | 0,962 | 0,981 | 1 | 12,398 | 0,902 | 0,928 | 0,962 | 1 | 16,673 |
| 4 | 1 | 16 | 0,959 | 0,967 | 0,981 | 1 | 20,382 | 0,922 | 0,936 | 0,963 | 1 | 25,637 |
| 5 | 1 | 25 | 0,965 | 0,970 | 0,981 | 1 | 30,390 | 0,933 | 0,943 | 0,964 | 1 | 36,598 |

Table 3 Factors $k_1$ and $k_2$ for various Support Conditions

For the above mentioned tie rod, the values of $f_1$ up to $f_5$ were calculated for support conditions no. 2, no. 3 and no. 4 ($f_1$ equal to $f_1$ from test data). The results are shown in Table 1, line 2, 3 and 4. These simplified frequency spectra show clearly, that the actual restraint value lies between support condition no. 3 and no. 4.

The tensile force as a function of $f_n$ can be obtained by using the following equation where $Z_{cable,n}$ is the tensile force using equation no. 3:

$$Z_n = k_1 \left[Z_{cable,n} - k_2 \cdot N_E\right] \tag{5}$$

If all values of $Z_n$ for support condition no. 2, 3 and 4 are determined from the test results, the results shown in Table 2 are obtained. All these values can be represented graphically (see Fig. 3).

## 4.2 FINAL EVALUATION OF TEST DATA

By considering Table 1 and Fig. 3 it is obvious that the actual support condition and the actual tie rod force Z lie somewhere between support condition no. 3 and no. 4.

Because there is only one single actual tensile force Z for all measured $f_n$ in the tie rod under consideration, the force Z must be a horizontal line between support condition no. 3 and no. 4 (see Fig. 3).

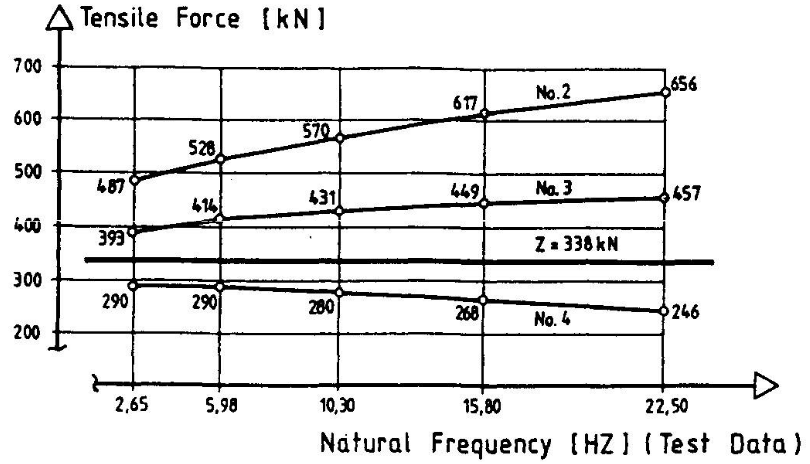


Fig. 3 Tensile Forces Z for various Support Conditions and actual Tie Rod Force

For each natural frequency, the unknown Z-force can be expressed as

$$Z_n = Z_{4,n} + \alpha \cdot (Z_{3,n} - Z_{4,n}) \tag{6}$$

The sum of the errors squared $D = \sum_{n=1}^{5} \left(Z_n - Z\right)^2$ (7) can be minimised by using

$$\frac{dD}{d\alpha} = 0 \quad \text{and} \quad \frac{dD}{dZ} = 0 \tag{8}$$

Two equations with two unknown quantities $\alpha$ and Z are thus obtained. The actual Z-force in the tie rod can then be assessed fairly accurately. In this case 338 kN.

Fortunately, after evaluating all the data, the measured forces were generally found to conform with the previously calculated and expected forces.

A structural concept for strengthening was then determined on this proven basis.

## 5. DESIGN CONCEPT FOR STRENGTHENING

In order to obtain the desired load reserves of 50 kN each per beam end, even under a full snow load of 0,75 kN/m$^2$, it was decided to reduce the weight of the roof cap.

The actual roofing, with a weight of 1,3 kN/m$^2$, was replaced by a construction weighing only 0,3 kN/m$^2$, by using trapezoidal steel sheeting and an insulating course. Some beams were also strengthened using stiffeners in areas where buckling was a problem.

After completion of the reinforcement work, the required load reserve of 50 kN per beam under full snow load was available, which is a total load reserve of 400 kN above hall stage, where approx. 8 beams meet.

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