Investigation and Simulation of Dynamic Behaviour of Railway Bridges with Ballast Substructure

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Abstracts: The dynamic investigation of railway bridges with ballast bed has been continued by the Institute of Structural Engineering – Research Center of Steel Structures. Now the results of the comparison of measurement and calculation with the already developed model for the ballast bed for railway bridges are shown for an operated bridge. These results demonstrate the practical application of the model. Nevertheless new detected dynamic effects on operated bridges have to be investigated furthermore for an exact dynamic modelling of the ballast substructure of railway bridges.

1. Introduction

A model describing the dynamic behaviour of the ballast substructure of railway bridges has been developed and adapted to an experimental bridge in laboratory ([Mähr 2009], [Hackl 2012], [Kirchhofer 2012], [Fink et al. 2013]). In figure 1 the experimental bridge with mounted vibration generators is shown. It is built up of two steel main beams and a wooden ballast trough with a standard ballast substructure and rails.

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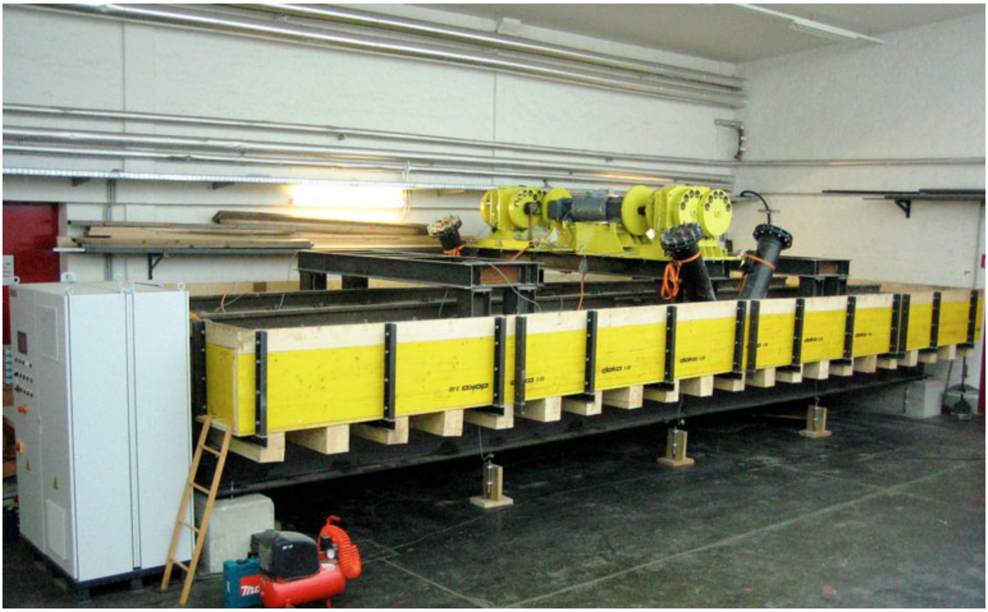

Figure 1: Experimental bridge in laboratory; [Mähr 2009]

To get an overview of the model it is described here in brief: The ballast bed takes part in the dynamic stiffness and the damping of railway bridges. This happens through connection of the rail and the bearing structure by the ballast bed. It transfers shear forces and so damping is activated. For modeling these effects, spring and damping elements (parameters k_b, c_b; fig. 2) are used.

Therefore the rail is diagonally connected with the bearing structure in a horizontal distance according to the distance of the sleepers (parameter e_s = 60 cm).

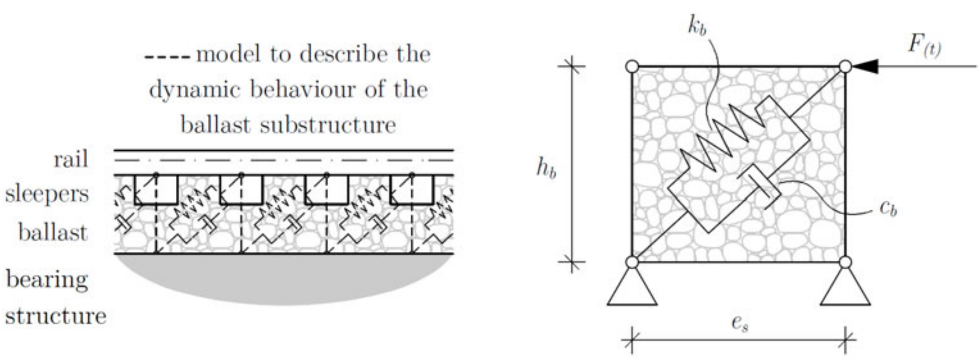

Figure 2: Model of the ballast substructure and detailed model with its parameters

Couplings fix the relative vertical displacement between rail and bearing structure and their parameter is marked with h_b (according to the height of the ballast bed, h_b = 50 cm).

The dynamic parameters k_b, c_b were detected by adaption of the computer model to measured results (frequency response) of the experimental bridge in laboratory by using two vibration generators to create a known dynamic excitation ($P_t = me\Omega^2 \sin \Omega t$). The product of me (eccentric mass m x eccentricity e) defines the value of excitation. The values of the parameters k_b, c_b for different excitation are given per rail in table 1.

Table 1: Parameters of the ballast substructure detected at the experimental bridge (excitation with two vibration generators), [Hackl 2012]

me	2×191	2×370	2×525	2×648	2×728	2×761	[kgcm]
k_b	14055	11345	9930	9080	8915	8915	[kN/m]
c_b	94,60	97,00	95,76	94,32	96,62	97,78	[kNs/m]

For the parameter k_b and the value of excitation me following correlation can be given:

$$k_b = 0{,}003(me)^2 - 11{,}61me + 1794 \tag{1}$$

The damping factor c_b has no correlation which could be given in an equation.

In this paper the behavior of an operated railway bridge is investigated. Therefore the calculated frequency response of an operated bridge is compared with the measured one. For the calculation the parameters k_b, c_b of the experimental bridge were used. This should show if the developed model is able to describe the dynamic behavior of the operated bridge by using the detected parameters of the experimental bridge or if there are any other effects which have to be considered in the modeling to get the same results on calculation and measurement.

2. Description of the investigated bridge

The used data for evaluating the computer model are from “Fahrenbachviadukt” near Persenbeug at Danube [Kirchhofer 2012]. This bridge (fig. 3) is part of an already closed section of the railroad near the Danube between St. Valentin and Krems in Lower Austria.

Figure 3: “Fahrenbachviadukt”; [Kirchhofer 2012]

It is a single span steel bridge with orthotropic plate and open cross-section which was built in 1995. In figure 4 the statically effective cross section of the bridge and the ballast substructure with wooden sleepers are shown.

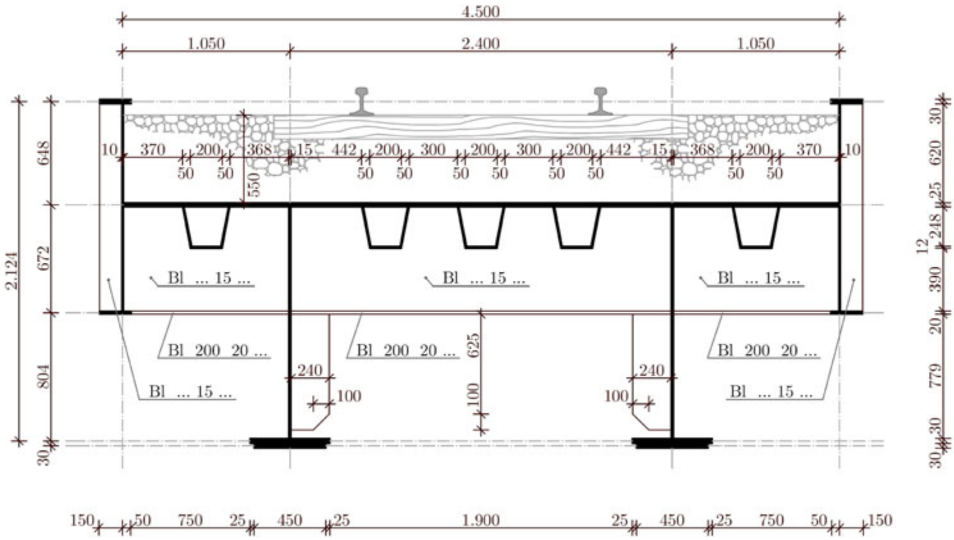

Figure 4: Cross Section of the “Fahrenbachviadukt”; [Hackl 2012]

The bearing construction has a span of 21,35 m and it is bedded on two roller bearings and on two line rocker bearings on the other river side. In the midspan steel plates are added to the lower flanges of the main beams to strengthen the bridge (fig. 4). Cantilever arms carrying the walkway and the cable through are mounted on the edge beams (fig. 3).

The measured data are frequency responses for different excitations me. These data are from dynamic tests and are given in table 2.

Table 2: Results of the measurement, [Hackl 2012]

me	2×110	2×180	2×244	2×271	2×326	2×363	[kgcm]
f	5,800	5,750	5,717	5,683	5,650	5,633	[Hz]
w_{max}	1,038	1,708	2,281	2,570	2,993	3,218	[mm]
a_{max}	1,441	2,464	3,059	3,518	4,272	4,712	[m/s²]

The average values are given. The first eigenfrequency *f* and for both main beams the displacement w_{max} and the acceleration a_{max} were measured. They had little differences caused by a different excitation of the bearing construction. This different excitation is caused by a slanted track through a curve in the area of the bridge.

3. Description of the used computer model

First a finite element model of the statically effective cross section was built (with SOFiSTiK FEM package). In the model the bearings were distributed to the nodes of the lower flange and the web of the main beams in area of the bearing to reduce the bearing reaction in the several single nodes. The nodes of the web are only fixed in vertical direction. This should prevent mistakes in numerical calculation. To this model the model of the ballast substructure was added (fig. 5). The detail in figure 5 shows the easy practice of the model of the ballast substructure by using given couplings (yellow), spring and damping elements (the spring includes both of them, green) of the finite element programme. For avoiding higher eigenmodes of the rails in the output of dynamic calculation they are fixed in the lateral direction. The dead load of the steel construction is already considered by the programme. Other loads like the masses of the ballast bed (20 kN/m³ for first calculation) and the sleepers are considered as surface load on bridge deck. Kirchhofer [Kirchhofer 2012] modified bogies (two axes) to mount the vibration generators for using them on rail tracks. They were used for exciting the bridge. Four point loads at the contact points in midspan per rail simulate the mass (2,0375 t per contact point) and the dynamic excitation ($P_{(t)}$) of these testing machines (fig. 5).

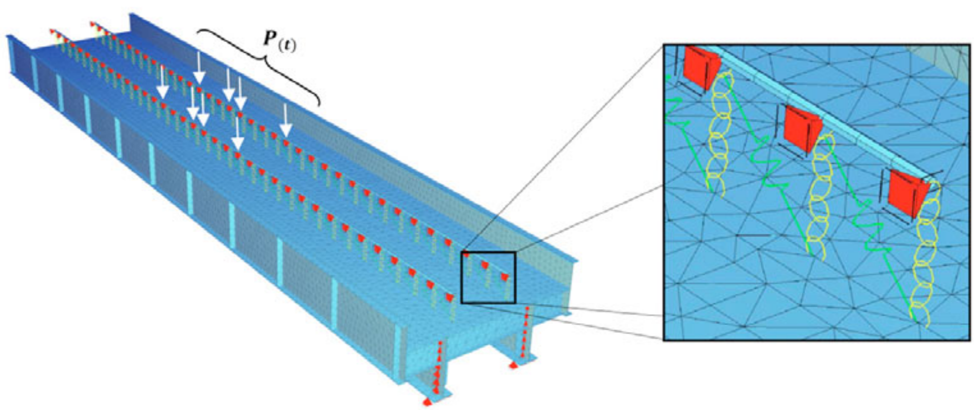

Figure 5: Finite element model of the “Fahrenbachviadukt” including the model of the ballast substructure

To consider the self damping of the steel construction and the bearings values of damping for railway bridges are used. They are given in three categories for three construction parts in PETERSEN [Petersen 1996]. He gives values for damping for low, middle and high vibration amplitude but no definition for these characteristics. So middle was chosen. The given value for the construction part “ballast” was set to zero because for that the detected parameters of the model (tab.1) from the testing bridge in laboratory were used.

4. Simulation of the dynamic behaviour of "Fahrenbachviadukt"

In the first step for the calculation the statically effective cross section (fig. 4) and a density of the ballast of 20 kN/m³, which is given in Eurocode 1 [EC 1 2003], were used. The parameters k_b for the spring elements were identified in equation (1) for the values of excitation me which were used for this bridge. A value of 95 kNs/m (tab. 1) was defined for the damping parameter c_b.

The result of this calculation shows a big difference from the measured dynamic values (tab.1) of the bridge. Especially the different first eigenfrequency results let assume that the unsimulated parts of the construction acquire stiffness which influences the eigenfrequency. So the cantilever arms with the walkway and the handrail were modelled. All configurations in table 3 are modelled like that.

Table 3: Results of the parameter study (me =2×110 kgcm) for ballast bed with 20,00 kN/m³ and comparison with measured results given as percentage deviation, [Hackl 2012]

k_b	c_b	$k_{b,d}$	$c_{b,d}$	c_{ap}	f	deviation	w_{max}	deviation
[kN/m]	[kNs/m]	[kN/m]	[kNs/m]	[kN/s]	[Hz]	[%]	[mm]	[%]
configuration 1								
15531	95				5,277	9,0	2,516	142,4
15531	150				5,277	9,0	1,922	85,2
∞	1000				5,505	5,1	4,875	369,7
configuration 2								
15531	95			95	5,277	9,0	0,990	4,6
∞	95			105	5,505	5,1	1,069	3,0
configuration 3								
15531	95	15531	95	95	5,405	6,8	0,858	17,3
100000	95	50000	95	105	5,711	1,5	1,117	7,6

The results were a bit better than the first ones but not satisfying. Then a parameter study for k_b and c_b was done which shows that the model with configuration 1 does not include all effects of dynamic behaviour. In table 3 this fact (configuration 1) and the following results for an excitation of 2x110 kgcm are overviewed. There, in the left five columns the used parameters for calculation are shown and in the four right columns the results (f, w_{max}) and their deviation to the measured results (see tab. 2) are shown. In configuration 2 a damping of the apron plate at the change of bridge deck to the dam is supposed. Therefore the parameter c_{ap} is established. This helps to simulate the displacement with higher accordance to the measured one. The eigenfrequency could not be reached even by setting the parameter k_b to infinity. To simulate the correct eigenfrequency the ballast substructure in front of and behind the bridge, bedded on the dam, was modelled (fig. 6). Therefore the parameters $k_{b,d}$ and $c_{b,d}$, are established and their values were set equal to the parameters on the bridge. This test is called configuration 3. It can be seen that this procedure does not help to reach the measured values because the parameters k_b (100000 kN/m) and $k_{b,d}$ (50000 kN/m) which afford a variation of 1,5 % are unrealistic high. So following consideration was done to reach the aim of a perfect dynamic modeling of the bridge.

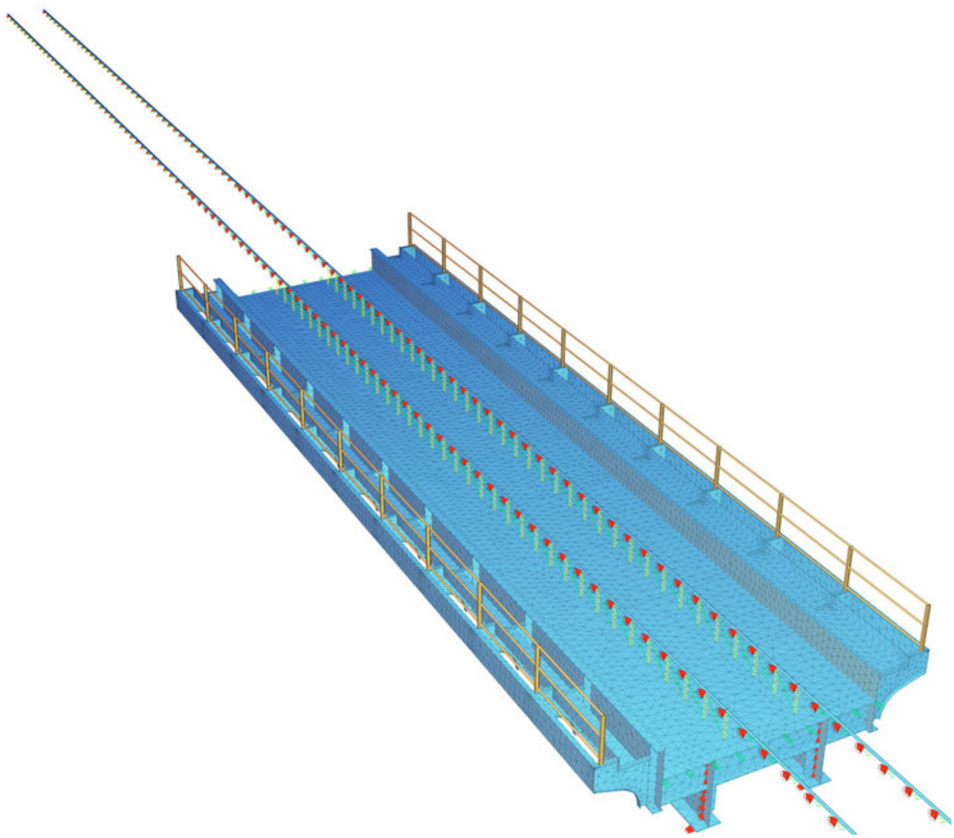

Figure 6: Finite element model of the “Fahrenbachviadukt” - configuration 3; [Hackl 2012]

The eigenfrequency depends on the stiffness and on the mass. A modification of the stiffness in the meaning of a more exact assessment of it in the finite element model did not bring the expected result. So the mass of the bridge must be the decisive factor. In the paper of GOTTSCHOL & KEMPFERT [Gotschol & Kempfert 2004] the answer of the problem was found. They give a value of density of basalt gravel for ballast bed of 17,25 kN/m³. This is an upper limit because basalt gravel is the heaviest in this category. The Austrian Federal Railways (ÖBB) set the density to 15,00 kN/m³. So the instruction of the standard with a density of 20,00 kN/m³ is much too high for a calculation of the eigenfrequency.

A new calculation with the corrected value of the density (17,25 kN/m³) of the ballast bed brought the expected perfect finish of the modeling (tab. 4).

A new test (configuration 4, table 4) with the model with statically effective cross section and the same conditions as configuration 3 (tab. 4) brings nearly the same results as configuration 3. So a modeling of construction details which are not included in the statically effective cross section seem to be needless and therefore time for calculation and modeling is saved. By using a density of 15,00 kN/m³ for ballast bed for configuration 4 modeling the bridge with hardly any deviation is possible (tab. 5). When the damping of the apron plate is switched off the displacement has a high deviation of about 47 % (configuration 5). So the dynamic behaviour of the apron plate seems to be necessary for modeling.

In table 6 a summary of the used configurations for helping the interpretation of table 3 to 5 is shown.

Table 4: Results of the parameter study (me =2×110 kgcm) for ballast bed with 17,25 kN/m³ and comparison with measured results given as percentage deviation, [Hackl 2012]

k_b	c_b	$k_{b,d}$	$c_{b,d}$	$c_{a,p}$	f	deviation	w_{max}	deviation
[kN/m]	[kNs/m]	[kN/m]	[kNs/m]	[kN/s]	[Hz]	[%]	[mm]	[%]
configuration 1								
15531	95				5,480	5,5	2,511	141,9
15531	200				5,480	5,5	1,550	49,3
configuration 2								
15531	95			84,5	5,480	5,5	1,038	0,0
configuration 3								
15531	95	15531	95	55	5,613	3,2	1,043	0,5
configuration 4								
15531	95	15531	95	56	5,638	2,8	1,038	0,0

Table 5: Results of the parameter study (me =2×110 kgcm) for ballast bed with 15,00 kN/m³ and comparison with measured results given as percentage deviation, [Hackl 2012]

k_b	c_b	$k_{b,d}$	$c_{b,d}$	$c_{a,p}$	f	deviation	w_{max}	deviation
[kN/m]	[kNs/m]	[kN/m]	[kNs/m]	[kN/s]	[Hz]	[%]	[mm]	[%]
configuration 4								
15531	95	15531	95	56	5,834	0,6	1,037	0,1
configuration 5								
15531	95	15531	95		5,834	0,6	1,527	47,1

Table 6: Summary of the used configurations

configuration 1	modeling of the ballast substructure on the bridge with complete cross section; parameters: k_b, c_b
configuration 2	modeling of the ballast substructure on the bridge and damping of the apron plate with complete cross section; parameters: k_b, c_b and $c_{a,p}$
configuration 3	modeling of the ballast substructure on the bridge, damping of the apron plate and of ballast substructure in front of and behind the bridge with complete cross section; parameters: k_b, c_b, $c_{a,p}$, $k_{b,d}$, $c_{b,d}$
configuration 4	modeling of the ballast substructure on the bridge, damping of the apron plate and of ballast substructure in front of and behind the bridge with statically effective cross section; parameters: k_b, c_b, $c_{a,p}$, $k_{b,d}$, $c_{b,d}$
configuration 5	modeling of the ballast substructure on the bridge and of ballast substructure in front of and behind the bridge with statically effective cross section; parameters k_b, c_b, $k_{b,d}$, $c_{b,d}$

5. Conclusions

In the investigations the possibility of modeling the ballast substructure on railway bridges for exact and easy dynamic calculation is shown. Therefore the knowledge of the correct density of the ballast bed is essential which is demonstrated in this paper. This is the deciding influence on the dynamic calculation. The second factor influencing the eigenfrequency of constructions, the stiffness, does not have that importance in calculation. So simplified modeling of investigated bridges is allowed where the modeling of the main construction is enough.

Nevertheless further investigations are necessary to get more informations about the dynamic behaviour of parts of the construction which seem to be essential for calculation. Now this behaviour of these features is assumed but the correct relationships among these factors are not known. The meant features are the damping of the apron plate with the parameter $c_{a,p}$, the stiffness and the damping ($k_{b,d}$, $c_{b,d}$) of the ballast substructure in front of and behind the bridge and at last correct damping properties of different types of bearing (k_B, c_B). Further the length of the ballast substructure L_d in front of and behind the bridge has to be detected. This new field of investigation in dynamics of railway bridges can be seen in figure 7 where the construction parts and its parameters are clearly represented.

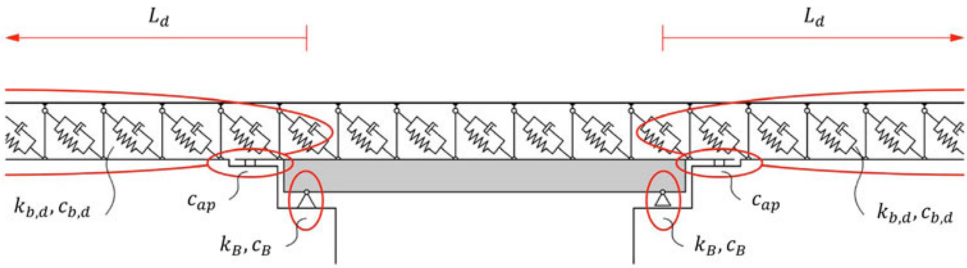

Figure 7: New parameters in dynamics of railway bridges

References

[Mähr 2009] Mähr, T.C., Theoretische und experimentelle Untersuchungen zum dynamischen Verhalten von Eisenbahnbrücken mit Schotteroberbau unter Verkehrslast, Phd thesis, TU Wien, 2009

[Hackl 2012] Hackl, K., Entwicklung von Rechenmodellen zur Beschreibung der dynamischen Eigenschaften des Schotteroberbaues bei Eisenbahnbrücken, Master thesis, TU Wien, 2012

[Kirchhofer 2012] Kirchhofer, J., Beitrag zur vertiefenden Beschreibung des dynamischen Verhaltens von Eisenbahnbrücken mit Schotteroberbau, Phd thesis, TU Wien, 2012

[Fink et al. 2013] Fink, J.; Kirchhofer J.; Hackl, K., Design of computer models for describing the dynamic behaviour of the ballast substructure of railway bridges, IABSE Conference – Assessment, Upgrading and Refurbishment of Infrastructure, Rotterdam, 2013

[Petersen 1996] PETERSEN, C., Dynamik der Baukonstruktionen, Braunschweig/ Wiesbaden, 1996

[EC 1 2003] ÖNORM EN 1991-1-1; Eurocode 1: Einwirkungen auf Tragwerke. Österreichisches Normungsinstitut, März 2003. Teil 1-1: Allgemeine Einwirkungen – Wichten, Eigengewicht, Nutzlasten im Hochbau

[Gotschol & Kempfert 2004] GOTSCHOL, A; KEMPFERT, H.-G., Zyklisch viskoelastisch-viskoplastischer Stoffansatz nichtbindiger Böden und Schotter, Bautechnik, 81(4), Berlin, 2004