Example 30: Verifying the interaction between two bridge piers

1 Description of the problem

In order to check the interaction between foundations, the determined settlements after manual calculation from *EWB* (2003), example 8.2, page 119 are compared with the results of the *ELPLA* program under two bridge piers.

As part of a new highway construction, a two-way bridge consisting of separate, parallel piers for each way direction will be built.

The clear distance between the pier foundations is 3.0 [m]; The foundation is shallow in the sloped deposits in a hanging situation. The governing modulus of compressibility for the deposits was set to E_s = 25 [MN / m^2]. The rock is considered uncompressible. As an alternative to a low-deformation deep foundation, this example examines the more economical design of a shallow foundation with regard to the foundation cost. In order to reduce the canting of the bridge piers, they are connected at a height of 25.0 [m] above the lower edge of the foundation by a bar. The geometry of the piers and the subsoil are shown in the section in Figure 69 and in Figure 70 in plan view.

The normal forces N occurring in the bar are wanted when placing the superstructures. The superstructure of the pier A should first be constructed for construction reasons. After completion of the directional lane A, the superstructure on pier B is constructed. In the example, only the load cases "constructing superstructures" are examined. In real terms, further load cases should be investigated.

The example task is to be solved with a calculation method that allows a hand calculation. This is done under the assumption that the bar is hinged to the piers and can be used for all components with respect to the normal and flexural stiffness $EA = EI = \infty$. Furthermore, it is assumed that a consideration of the characteristic points is sufficiently accurate for the description of the foundation settlement. The dead load of a superstructure per pier is 25 [MN].

2 Calculation method according to *EWB* (2003)

The calculation is carried out according to the direct stiffness method as a manual calculation using a stress determination in the characteristic points ($K1$ and $K2$) of the pier foundations (A and B). Both the stresses arising from the load on the foundation itself and the influence of the respective neighboring foundation must be taken into account. The settlements then result from the integration of the stresses up to the respective effective depth (= upper rock edge). The foundations are first calculated without consideration of the bar. From this calculation results a convergence of the piers, which must be prevented by the bar. From this consideration, the normal force of the bar can be calculated.

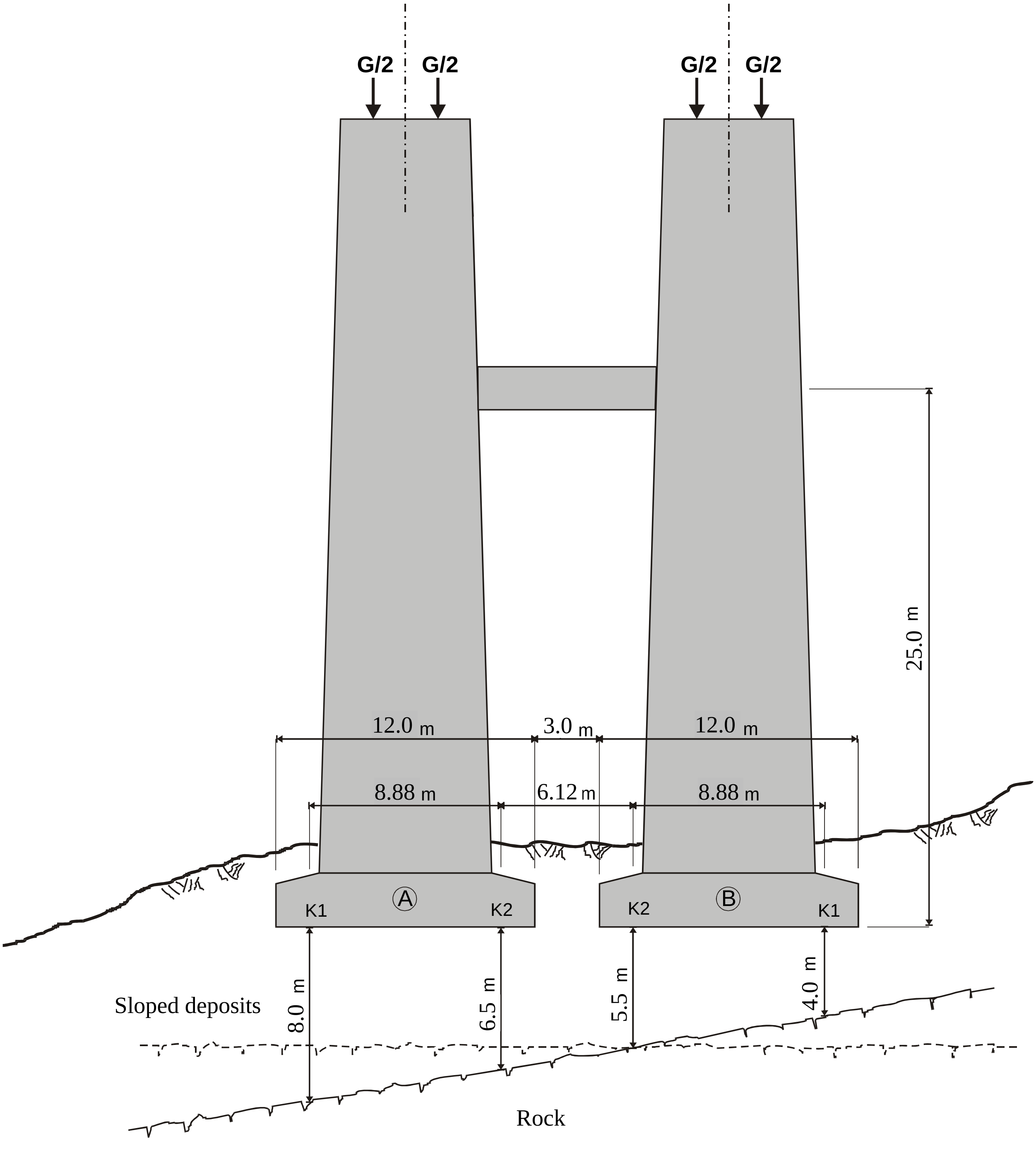

Figure 69 A section represents the geometry and subsoil

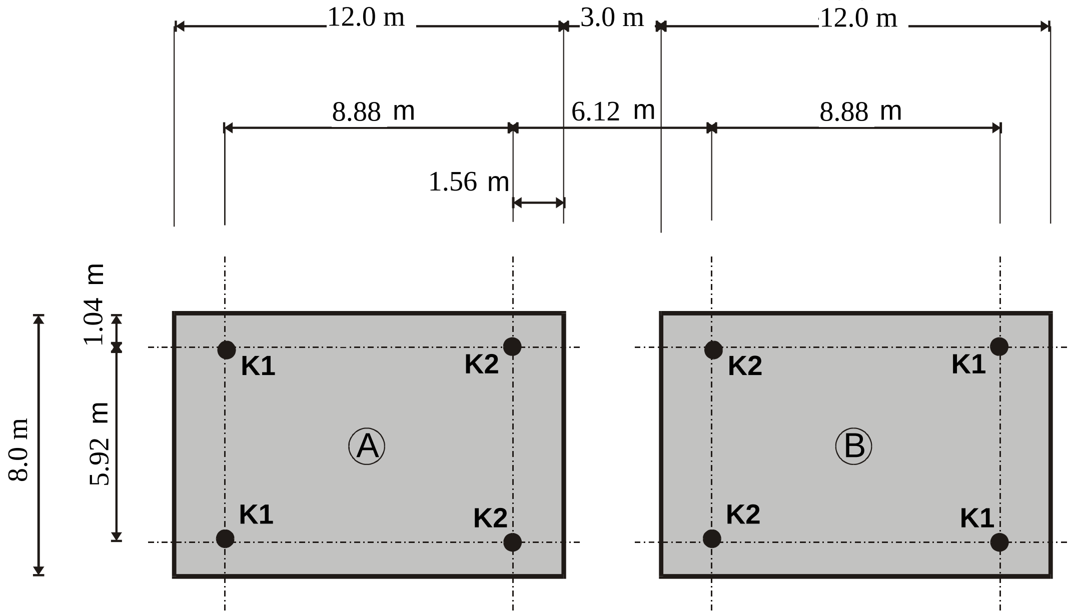

Figure 70 A plan view showing the characteristic points K1 (the side away from the neighboring pier) and K2 (the side facing the neighboring pier)

3 Hand calculation

3.1 Settlement of the piers due to dead load of the superstructures

The determination of the settlements in the characteristic points is carried out with a mean average normal stress:

$$q = 25\ [\text{MN}] / (12\ [\text{m}] \times 8\ [\text{m}]) = 0.2604\ [\text{MN/ m}^2]$$

The calculation of i takes place according to Eq. 9.5 and 9.6 of the *EVB* (1993) with $i = \sigma_z / q$. The location of the characteristic point and the division of the area into subareas 1 to 4 are given in Figure 71 (for dimensions, see Figure 70 and Figure 71).

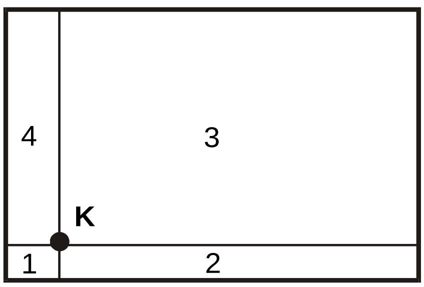

Figure 71 Location of the characteristic point and subareas 1 to 4

Table 41 Calculation and integration of the stresses at the characteristic point due to dead load for piers *A* and *B*

Depth below edge of foundation z [m]	Area 1 a =1.56 [m] b =1.04 [m]		Area 2 a =10.44 [m] b =1.04 [m]		Area 3 a =10.44 [m] b =6.96 [m]		Area 4 a =6.96 [m] b =1.56 [m]		Stress σ_z [kN/m^2]	Int. of stress $\int \sigma_z\, dz$ [kN/m]
	q =0.2604 [kN/m^2]									
	z/b	i	z/b	i	z/b	i	z/b	i		
0.000	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.250	0.260	0.000
0.500	0.481	0.239	0.481	0.241	0.072	0.250	0.321	0.247	0.254	0.129
1.000	0.962	0.198	0.962	0.208	0.144	0.250	0.641	0.232	0.231	0.250
1.500	1.442	0.150	1.442	0.171	0.216	0.249	0.962	0.207	0.202	0.358
2.000	1.923	0.112	1.923	0.141	0.287	0.247	1.282	0.182	0.178	0.453
2.500	2.404	0.085	2.404	0.119	0.359	0.245	1.603	0.159	0.158	0.537
3.000	2.885	0.065	2.885	0.102	0.431	0.242	1.923	0.140	0.143	0.613
4.000	3.846	0.041	3.846	0.079	0.575	0.233	2.564	0.110	0.120	**0.744**
5.500	5.288	0.023	5.288	0.058	0.790	0.214	3.526	0.080	0.098	**0.908**
6.500	6.250	0.017	6.250	0.048	0.934	0.200	4.167	0.066	0.086	**1.000**
8.000	7.692	0.012	7.692	0.038	1.149	0.178	5.128	0.051	0.073	**1.119**

it follows:

$$S_{K1,A} = 1.119 \text{ [MN/m]}/ 25 \text{ [MN}^2\text{]} = 4.477 \times 10^{-2} \text{ [m]}$$

$$S_{K2,A} = 1.000 \text{ [MN/m]}/ 25 \text{ [MN}^2\text{]} = 4.000 \times 10^{-2} \text{ [m]}$$

$$S_{K2,B} = 0.908 \text{ [MN/m]}/ 25 \text{ [MN}^2\text{]} = 3.632 \times 10^{-2} \text{ [m]}$$

$$S_{K1,B} = 0.744 \text{ [MN/m]}/ 25 \text{ [MN}^2\text{]} = 2.976 \times 10^{-2} \text{ [m]}$$

Annotation:
The canting of the pier *B* is greater than that of pier *A*, although the settlements of *A* are greater than that of *B*. This is because the compressible layer below the pier *A* is big, but the differential settlement between the characteristic points for *B* is higher.

3.2 Settlement of the piers due to neighboring effect

Stress determination at the characteristic points K1:

The stress result from the following superposition:

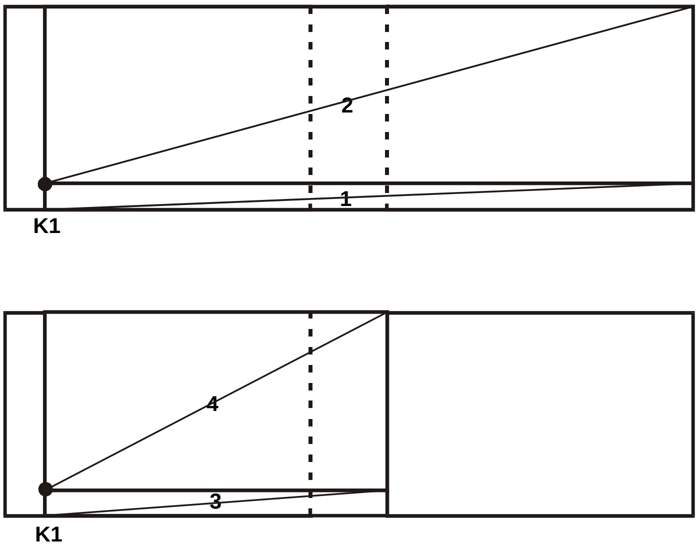

Figure 72 Superposition for stress determination at the characteristic points K1

Table 42 Calculation and integration of the stresses at the characteristic points K1 due to neighboring effect

Depth below edge of foundation	Area 1		Area 2		Area 3		Area 4		Stress	Int. of stress
	a =25.44 [m] *b* =1.04 [m]		*a* =25.44 [m] *b* =6.96 [m]		*a* =13.44 [m] *b* =1.04 [m]		*a* =13.44 [m] *b* =6.96 [m]			
z	*q* =0.2604 [kN/m^2]				*q* =-0.2604 [kN/m^2]				σ_z	$\int \sigma_z\, dz$
[m]	*z/b*	*i*	*z/b*	*i*	*z/b*	*i*	*z/b*	*i*	[kN/m^2]	[kN/m]
0.000	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.000
0.500	0.481	0.241	0.072	0.250	0.481	0.241	0.072	0.250	0.000	0.000
1.000	0.962	0.208	0.144	0.250	0.962	0.208	0.144	0.250	0.000	0.000
1.500	1.442	0.171	0.216	0.249	1.442	0.171	0.216	0.249	0.000	0.000
2.000	1.923	0.141	0.287	0.248	1.923	0.141	0.287	0.248	0.000	0.000
2.500	2.404	0.119	0.359	0.246	2.404	0.119	0.359	0.245	0.000	0.000
3.000	2.885	0.102	0.431	0.243	2.885	0.102	0.431	0.243	0.000	0.000
4.000	3.846	0.079	0.575	0.236	3.846	0.079	0.575	0.234	0.000	0.000
5.500	5.288	0.059	0.790	0.221	5.288	0.058	0.790	0.218	0.001	0.001
6.500	6.250	0.050	0.934	0.209	6.250	0.049	0.934	0.205	0.001	0.002
8.000	7.692	0.041	1.149	0.192	7.692	0.040	1.149	0.186	0.002	0.005

Stress determination at the characteristic points K2:

The determination is made as in points K1, but with the lengths 16.56 [m] (instead of 25.44 [m]) and 4.56 [m] (instead of 13.44 [m]).

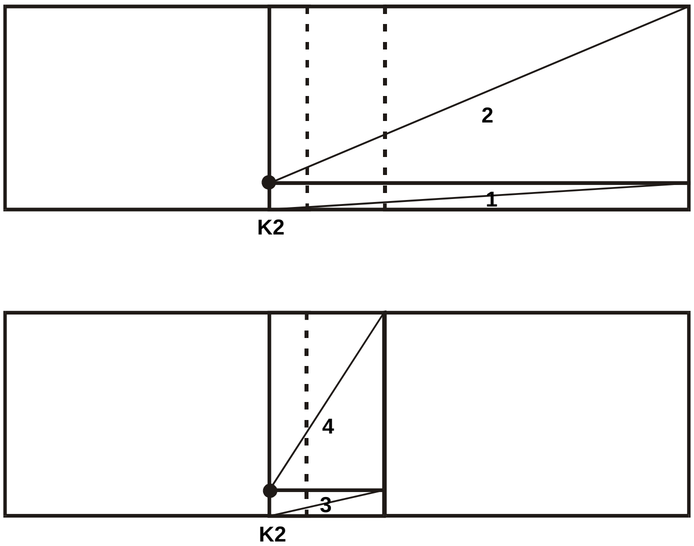

Figure 73 Superposition for stress determination at the characteristic points K2

Table 43 Calculation and integration of the stresses at the characteristic points K2 due to neighboring effect

Depth below edge of foundation	Area 1 *a* =16.56 [m] *b* =1.04 [m]		Area 2 *a* =16.56 [m] *b* =6.96 [m]		Area 3 *a* =4.56 [m] *b* =1.04 [m]		Area 4 *a* =6.96 [m] *b* =4.56 [m]		Stress	Int. of stress
	q =0.2604 [kN/m^2]				q =-0.2604 [kN/m^2]					
z [m]	z/b	i	z/b	i	z/b	i	z/b	i	σ_z [kN/m^2]	$\int \sigma_z\, dz$ [kN/m]
0.000	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.000
0.500	0.481	0.241	0.072	0.250	0.481	0.241	0.110	0.250	0.000	0.000
1.000	0.962	0.208	0.144	0.250	0.962	0.207	0.219	0.249	0.000	0.000
1.500	1.442	0.171	0.216	0.249	1.442	0.170	0.329	0.246	0.001	0.000
2.000	1.923	0.141	0.287	0.248	1.923	0.140	0.439	0.241	0.002	0.001
2.500	2.404	0.119	0.359	0.246	2.404	0.116	0.548	0.235	0.004	0.003
3.000	2.885	0.102	0.431	0.243	2.885	0.098	0.658	0.226	0.005	0.005
4.000	3.846	0.079	0.575	0.235	3.846	0.072	0.877	0.206	0.009	0.012
5.500	5.288	0.059	0.790	0.220	5.288	0.049	1.206	0.173	0.015	0.030
6.500	6.250	0.050	0.934	0.208	6.250	0.038	1.425	0.153	0.017	0.046
8.000	7.692	0.040	1.149	0.189	7.692	0.028	1.754	0.125	0.020	0.074

This results in the following settlements from neighboring effect:

$$S_{K1,A} = 0.005\ [\mathrm{MN/m}]/\ 25\ [\mathrm{MN^2}] = 0.020 \times 10^{-2}\ [\mathrm{m}]$$

$$S_{K2,A} = 0.046\ [\mathrm{MN/m}]/\ 25\ [\mathrm{MN^2}] = 0.184 \times 10^{-2}\ [\mathrm{m}]$$

$$S_{K2,B} = 0.030\ [\mathrm{MN/m}]/\ 25\ [\mathrm{MN^2}] = 0.120 \times 10^{-2}\ [\mathrm{m}]$$

$$S_{K1,B} \approx 0$$

3.3 Compilation of the results of the settlement calculations

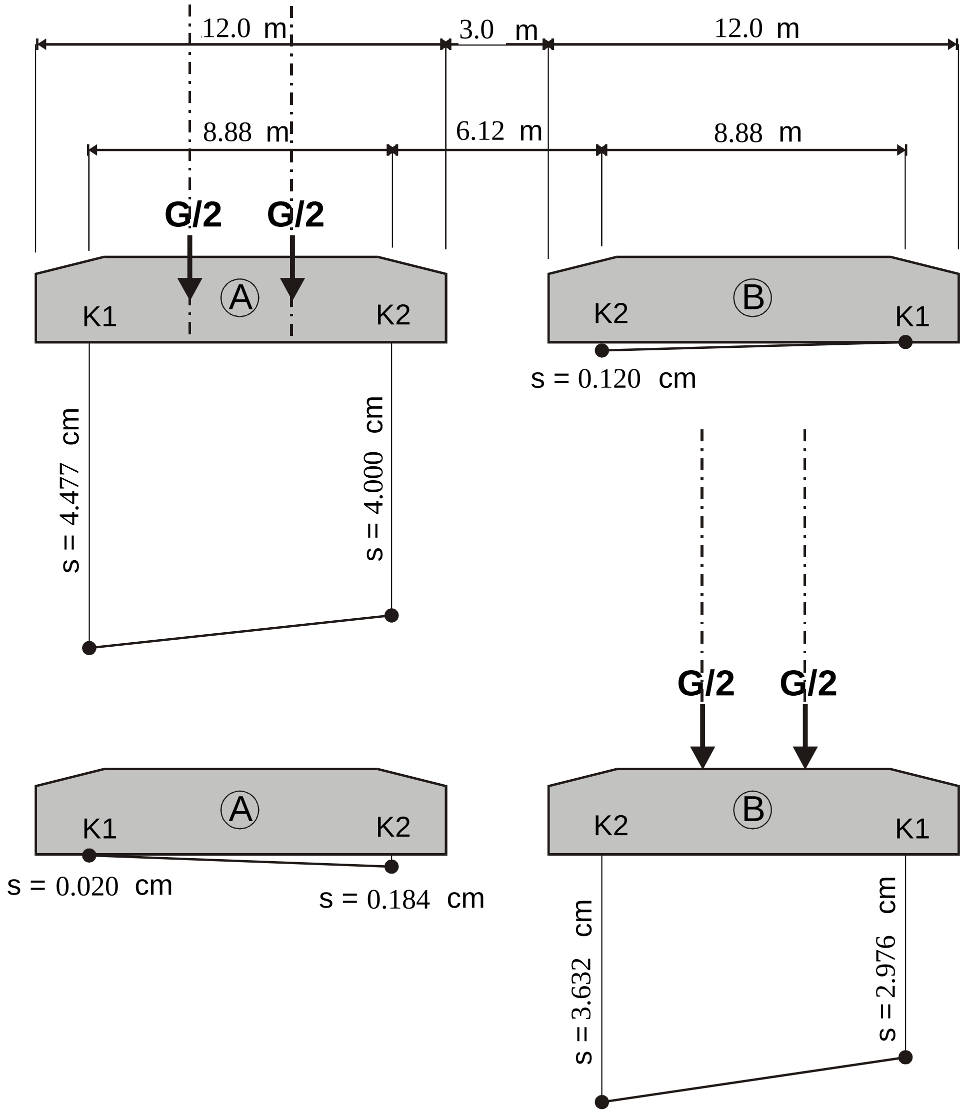

Figure 74 Settlement pattern for the foundations due to the superstructure load cases *A* and *B*

The settlement calculations carried out so far do not take into account the stiffness effect of the bar. By neglecting the normal force in the bar, this would result in the settlement pattern shown in section 8.2.6 for the foundations that are assumed to be rigid.

4 Calculation with ELPLA

4.1 Selection of the calculation method

The calculation is done using the iterative method of *Kany* / *El Gendy* (1997) for the rigid plate interactive system in the *ELPLA* program.

The calculation of the plates can be performed by iteration using two independent networks for the plate *A* and the plate *B*. The two plates are subdivided into 384 square elements each with a side length of 0.5 [m] (Figure 75).

In order to perform the calculation of the plates, two independent filenames of the two plates are selected.

The data is similar for the two plates except the origin coordinates.

The origin coordinates are chosen as (x_o, y_o) = (0.0, 0.0) for the plate *A* and (x_o, y_o) = (15.0, 0.0) for the plate *B*.

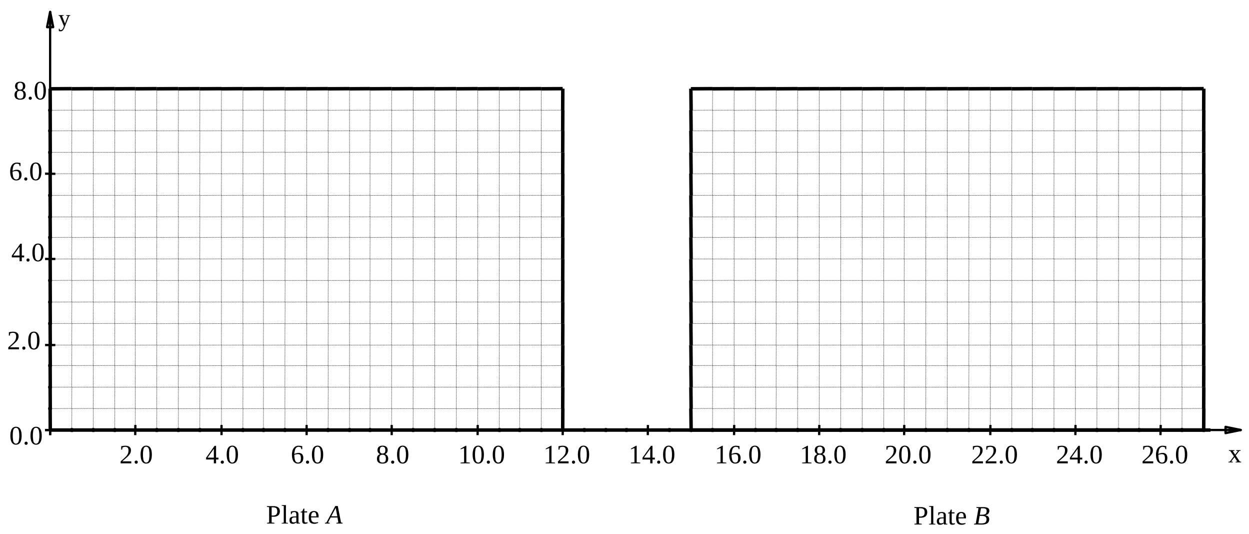

Figure 75 Plate A and B are built side by side

4.2 Consideration of the Boring points for different subsoil

From the available information about the subsoil, the subsoil profiles *B*1 to *B*4 have been developed as representative for the plate field. In order to be able to assign appropriate layer thicknesses to the individual elements of the field, the total area is subdivided into subareas as shown in Figure 76, with each subarea being assigned to one of the boring profiles. The allocation of the boring profiles to the individual nodes of the field surface divided into elements in the *ELPLA* program is shown in Figure 76.

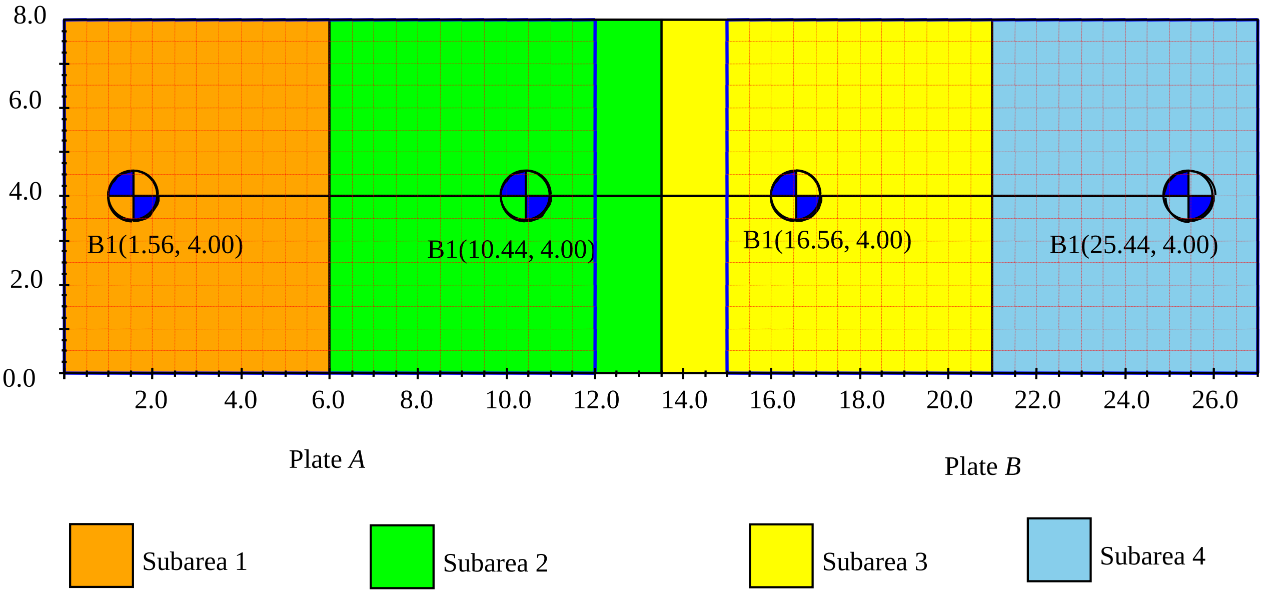

Figure 76 Location of the boring profiles *B*1 to *B*4 with allocation to the fields

5 Comparison of the results

The results for calculating the interaction between two bridge piers with the two different calculation assumptions by hand calculation from *EWB* (2003) and *ELPLA* are presented in tabular form.

Table 44 and Figure 77 show that there is no differential settlement.

Table 44 Settlements from *EWB* (2003) and *ELPLA*

Calculation	Settlement [cm]			
	Superstructure *A*		Superstructure *B*	
	*K*1	*K*2	*K*2	*K*1
EWB (2003)	4.497	4.187	3.752	2.976
ELPLA	4.219	3.982	3.632	2.529

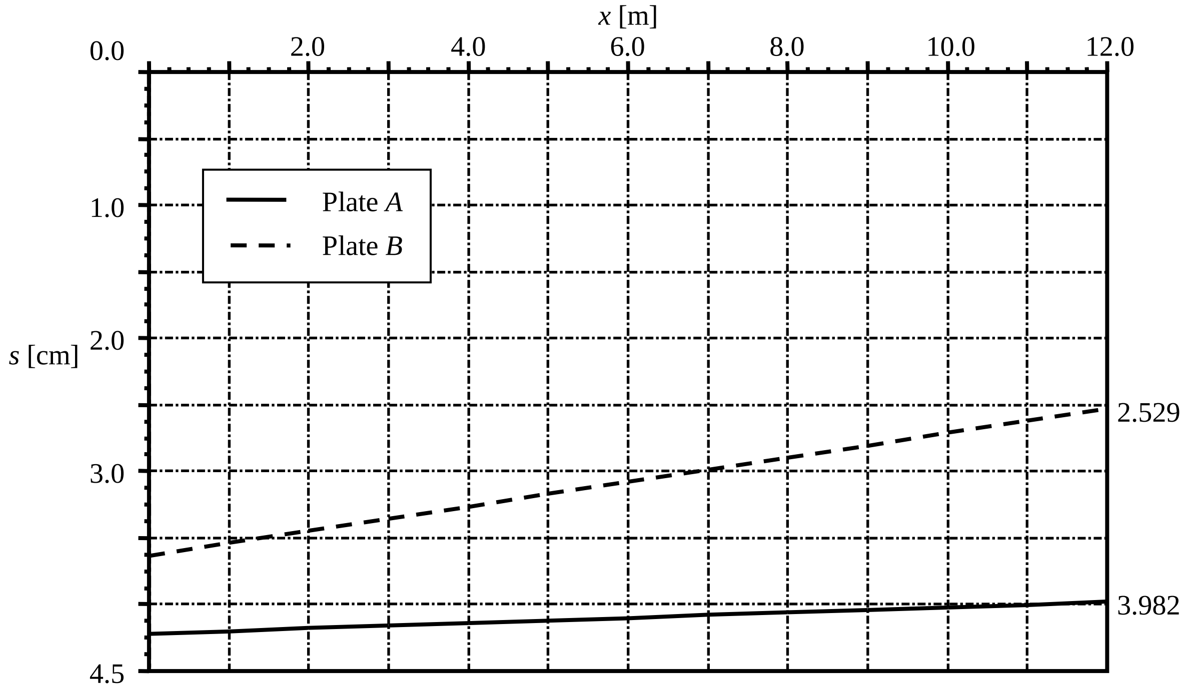

Figure 77 Settlements from *EWB* (2003) and *ELPLA*