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Autor: Maeda, Yukio

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A Design Method and Limit States for Pedestrian Steel Overpasses

YUKIO MAEDA
Dr.-Eng.
Professor of Civil Engineering
Osaka University
Osaka, Japan

Since the loading conditions for pedestrian steel overpasses is relatively simple and clear, the load-factor approach could be applied, which is practically simple and adequate for providing a safety concept and introducing probability into the design in a rational form.

On the other hand, the design criteria for main structures will be the limit of structural usefulness. If stocky sections are used for structural members, their plastic strength combined with stability limit may be the limit of usefulness, taking account of deflection and stress limits at the normal service loads or at the time of earthquake.

Here, design examples and their some results will be shown. The load factors for three different load combinations adopted at the Standard Rules for Plastic Design in Steel, Japan Welding Engineering Society, 1967, are as follows: (1) $U = 1.2\,D + 2.1\,L$ or $1.4\,(D + L)$, (2) $U = 1.2\,D + 1.7\,L + 1.7\,W$, (3) $U = D + L + 1.5\,E$, where U is ultimate strength, D is dead load, L is live load, W is wind load or snow load, and E is earthquake force or collision load. These values of the load factor were determined by a semi-probabilistic method.

Fig. 1 illustrate typical three types of pedestrian steel overpasses in Japan, and classifications of the types result in 44 different design cases, by span length which is 17.5 m or 22.0 m, by floor slab which is either reinforced concrete slab or steel deck, and by section of main structural members which is welded built-up or H-shaped rolled, and either uniform or non-uniform.

In proportioning the structural members for each case, the simple plastic theory was applied to the mechanism collapse as shown in Fig. 2. Also, secondary effects such as shear force, axial force, bucklings were considered, and the design of each case was done automatically by a computer. Particularly, in order to get a minimum weight of the members, the linearized relations between full-plastic moment and weight were applied to the calculation. Furthermore, an alternating collapse and an incremental collapse, and stress and deflection limits at the normal service load or at the time of earthquake, were investigated.

One example of the results is indicated in Table 1, which is for C Type, Portal Rigid Frames with variable sections of the members. The table shows that an increase of plastic moment due to the incremental collapse is about 10% for symmetrical form and 7% for anti-symmetrical one, and that an effect due to alternating plasticity can be neglected, but the design criteria is governed by the specified working stress at the normal service loads. If the live load is larger, the deflection may be the governing limit state.

Throughout the overall results, it is shown that the design criteria are the plastic strength due to mechanism collapse, or the incremental collapse, or the working stress or deflection which is to be specified at a rule or code, and that a priority among them depends upon the ratio of live load to total load, and upon the spanratio of the frame. If the values of load factors are changed, there will be different results of design criteria.

Finally, it may be said that load factors and limit states should be combined more rationally and in detail.

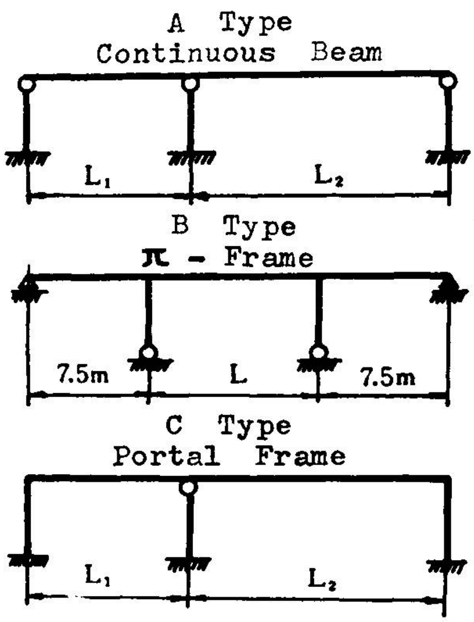
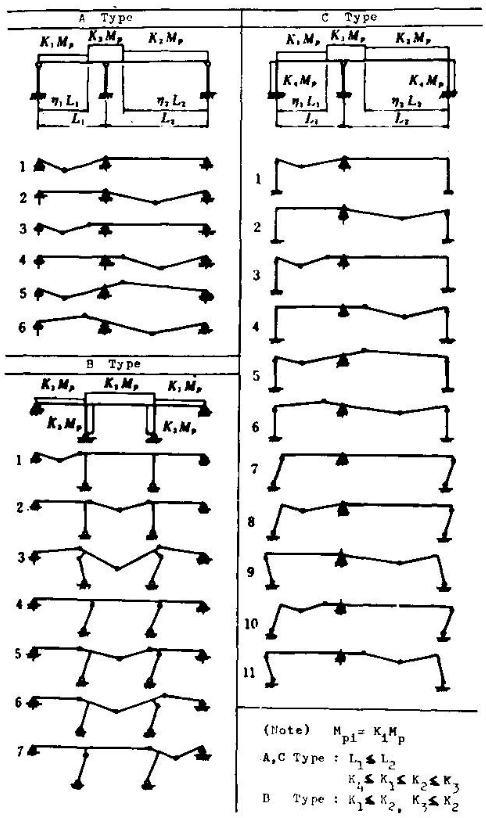


Fig. 1



Collapse mechanisms

Fig. 2

Table 1

C Type(variable section)

Item		Case	CVB 1	CVB 2	CVB 3	CVB 4	CVB 5	CVB 6
Load	D		330	330	330	500	500	500
	L		263	263	263	263	263	263
	EL		75	75	75	75	75	75
Location of variable section			0.9	0.8	0.8	0.9	0.8	0.9
Initial M_p ratio	K_1	K_3	1.0 / 1.0	1.0 / 2.2	1.0 / 1.0	1.0 / 1.0	1.0 / 2.2	1.0 / 1.0
	K_2	K_4	1.4 / 0.3	2.2 / 0.4	2.0 / 0.3	1.6 / 0.4	2.2 / 0.5	1.5 / 0.3
Final M_p ratio	K_1	K_3	1.0 / 1.0	1.0 / 1.0	1.0 / 1.0	1.0 / 1.0	1.0 / 1.0	1.0 / 1.0
	K_2	K_4	1.0 / 0.37	1.0 / 0.24	1.22 / 0.24	1.24 / 0.54	1.0 / 0.31	1.39 / 0.94
$_aM_p$		(t-m)	21.77	35.92	33.80	23.61	42.55	32.32
$_bM_p$		(t-m)	24.50	38.53	38.42	26.06	45.09	35.32
$_bM_p/_aM_p$			1.13	1.07	1.13	1.10	1.06	1.09
$_cM_p$		(t-m)	10.87	14.58	18.02	10.27	15.20	14.41
$_cM_p/_cM_p$			0.50	0.41	0.53	0.44	0.36	0.45
Section	section 1 (I-shaped)		200×12, 376×8	200×12, 536×8	200×12, 536×8	200×12, 376×8	200×12, 576×8	200×12, 476×8
	section 2 (I- shaped)		200×12, 376×8	200×8, 536×8	200×12, 536×8	200×12, 376×8	200×12, 576×8	200×12, 476×8
	section 3 (I-shaped)		200×12, 376×8	200×8, 536×8	200×16, 518×8	200×16, 380×8	200×12, 576×8	200×19, 462×8
	section 4 (box shaped)		200×8, 184×8	200×8, 184×8	200×8, 184×8	200×9, 232×8	200×9, 232×8	200×16, 318×9
Floor beam spacing (m)			1.50	1.40	1.40	1.50	1.40	1.40
Working stress (kg/cm^2)	at section 1		1 135	725	1 234	1 262	734	1 451
	at section 2		1 135	1 338	1 234	1 262	1 443	1 451
	at section 3		1 813	1 605	1 727	1 850	1 871	1 815
	* at section 4		1 979	2 153	3 346	2 160	2 374	1 581
Deflection		δ/L	1/409	1/425	1/415	1/449	1/538	1/443
Column t=9.5			609.6φ	609.6φ	609.6φ	609.6φ	609.6φ	609.6φ

(Note) * stress at earthquake