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PAVEMENT DESIGN DATA HAND BOOK

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2011

CONTENTS

1. ANALYSIS AND DESIGN OF FLEXIBLE PAVEMENT

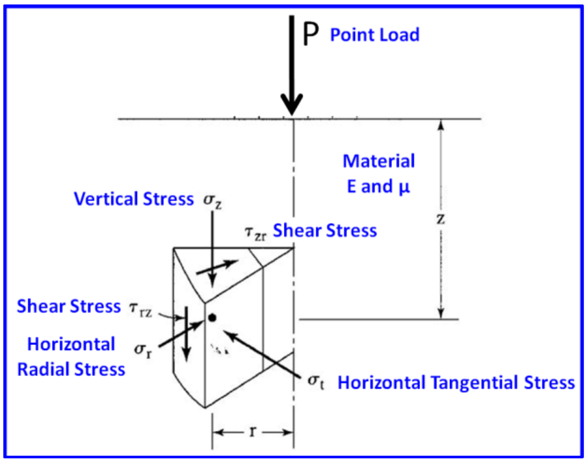
1.1 Boussinesq's Solution – Point Load

$\sigma_z = Vertical\ normal\ stress$

$\sigma_r = Radial\ normal\ stress\ (Horizontal)$

$\sigma_t = Tangential\ normal\ stress\ (Horizontal)$

$\tau_{zr} = Horizontal\ Shear\ stress\ (radial\ direction)$



The corresponding strains are:

$\epsilon_z = Vertical\ normal\ strain$

$\epsilon_r = Radial\ normal\ strain\ (Horizontal)$

$\epsilon_t = Tangential\ normal\ strain\ (Horizontal)$

$\gamma_{zr} = Horizontal\ Shear\ strain\ (radial\ direction)$

In defining these strains, the displacement field considered is two-dimensional; that is, a point in this space can move only vertically or horizontally, denoted by w and u, respectively.

The closed-form solution for a point load on an elastic half-space was originally developed by Boussinesq, Circa and adapted by Taylor in the following form:

$$\sigma_z = \frac{P}{2\pi}\frac{3z^3}{(r^2+z^2)^{5/2}} \quad (1.1\ a)$$

$$\sigma_r = \frac{P}{2\pi}\left[\frac{3r^2 z}{(r^2+z^2)^{5/2}} - \frac{1-2\mu}{r^2+z^2+z\sqrt{r^2+z^2}}\right] \quad (1.1\ b)$$

$$\sigma_t = \frac{P}{2\pi}(1-2\mu)\left[\frac{z}{(r^2+z^2)^{3/2}} - \frac{1}{r^2+z^2+z\sqrt{r^2+z^2}}\right] \quad (1.1\ c)$$

$$\tau_{zr} = \frac{P}{2\pi}\frac{3rz^2}{(r^2+z^2)^{5/2}} \quad (1.1\ d)$$

Note that for normal stresses, the sign notation is positive for tension and negative for compression. Note also that directly under the point of load application, (i.e., r = 0, z = 0), the stresses are undefined. The strain components are calculated from the stress components through generalized Hooke's law as:

$$\varepsilon_z = \frac{1}{E}[\sigma_z - \mu(\sigma_r + \sigma_t)] \quad (1.2\ a)$$

$$\varepsilon_r = \frac{1}{E}[\sigma_r - \mu(\sigma_z + \sigma_t)] \tag{1.2 b}$$

$$\varepsilon_t = \frac{1}{E}[\sigma_t - \mu(\sigma_z + \sigma_r)] \tag{1.2 c}$$

$$\gamma_{zr} = \frac{2\tau_{zr}(1+\mu)}{E} = \frac{\tau_{zr}}{G} \tag{1.2 d}$$

where **G** is the shear modulus of the elastic medium.

The vertical and horizontal deflections, ***w*** and ***u***, at any point are computed by integrating the vertical and horizontal strains respectively. The resulting expressions are:

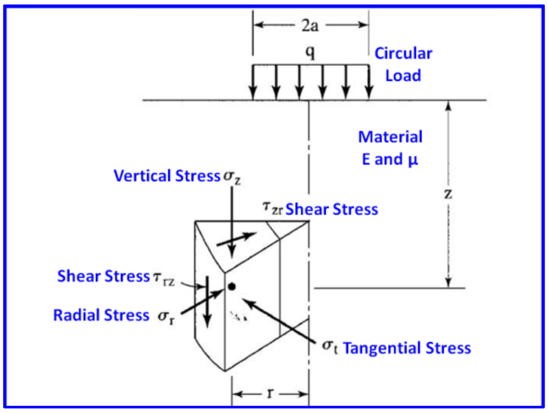
$$w = \frac{P}{2\pi E}\left[\frac{(1+\mu)z^2}{(r^2+z^2)^{3/2}} + \frac{2(1-\mu^2)}{\sqrt{(r^2+z^2)}}\right] \tag{1.3 a}$$

$$u = \frac{P(1+\mu)(1-2\mu)}{2\pi r E}\left[\frac{r^2 z}{(1-2\mu)(r^2+z^2)^{3/2}} + \frac{z}{\sqrt{(r^2+z^2)}} - 1\right] \tag{1.3 b}$$

1.2 Circular Load with Uniform Vertical Pressure

1.2.1 Solutions at Axis of Symmetry

The response due to a circular load with a radius a and uniform pressure q on an elastic homogeneous half-space is obtained by integrating the Boussinesq's components due to a concentrated load presented in the previous section. When the load is applied over a single circular loaded area, the most critical stress, strain, and deflection occur under the center of circular area on the axis of symmetry, where $\tau_{zr} = 0$ and $\sigma_r = \sigma_t$, so σ_z and σ_r are the principal stresses.



For points on the centerline of the load (i.e., **r = 0**), these stress components are given by

$$\sigma_z = q\left[1 - \frac{z^3}{(a^2+z^2)^{3/2}}\right] \tag{1.4 a}$$

$$\sigma_r = \sigma_t = \frac{q}{2}\left[(1+2\mu) - \frac{2(1+\mu)z}{\sqrt{a^2+z^2}} + \frac{z^3}{(a^2+z^2)^{3/2}}\right] \tag{1.4 b}$$

$$\tau_{zr} = 0 \tag{1.4 c}$$

Corresponding strain components are:

$$\varepsilon_z = \frac{(1+\mu)q}{E}\left[(1-2\mu) + \frac{2\mu z}{\sqrt{a^2+z^2}} - \frac{z^3}{(a^2+z^2)^{3/2}}\right] \quad (1.5\text{ a})$$

$$\varepsilon_r = \frac{(1+\mu)q}{2E}\left[(1-2\mu) - \frac{2(1-\mu)z}{\sqrt{a^2+z^2}} + \frac{z^3}{(a^2+z^2)^{3/2}}\right] \quad (1.5\text{ b})$$

and the vertical deflection under the centerline of the load is given by

$$w = \frac{(1+\mu)qa}{E}\left\{\frac{a}{\sqrt{(a^2+z^2)}} + \frac{1-2\mu}{a}\left[\sqrt{a^2+z^2} - z\right]\right\} \quad (1.6\text{ a})$$

$$w = \frac{3qa^2}{2E\sqrt{a^2+z^2}} \qquad \text{when } \mu = 0.5 \quad (1.6\text{ b})$$

On the surface of the half-space (i.e., z = 0)

$$w = qa\left[\frac{2(1-\mu^2)}{E}\right] \qquad \text{when } z = 0 \quad (1.6\text{ c})$$

1.2.2 Foster and Ahlvin Charts

Foster and Ahlvin (1954) presented charts for determining tangential stress **σ_t**, vertical stress **σ_z**, radial stress **σ_r**, shear stress **τ_{zr}**, and vertical deflection **w**, as shown in Figures 2.1 through 2.5. The load is applied over a circular area with radius a and an intensity q. Because Poisson ratio has relatively small effect on stresses and deflections, Foster and Ahlvin assumed the half-space to be incompressible with a Poisson ratio of 0.5, so only one set of charts is needed instead of one for each Poisson ratio.

Note : In the charts, $x-axis \rightarrow \frac{stress}{q}\,100 \qquad y-axis \rightarrow \frac{z}{a} \qquad No.\,on\,the\,Curve \rightarrow \frac{r}{a}$

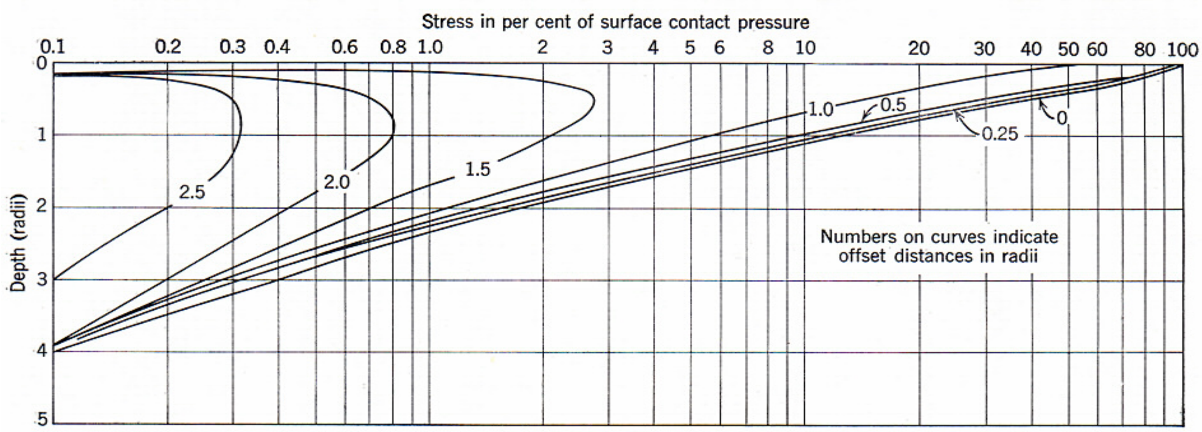


Figure 1.1 : Tangential Stress σ_t due to Circular Loading (Foster and Ahlvin, 1954)

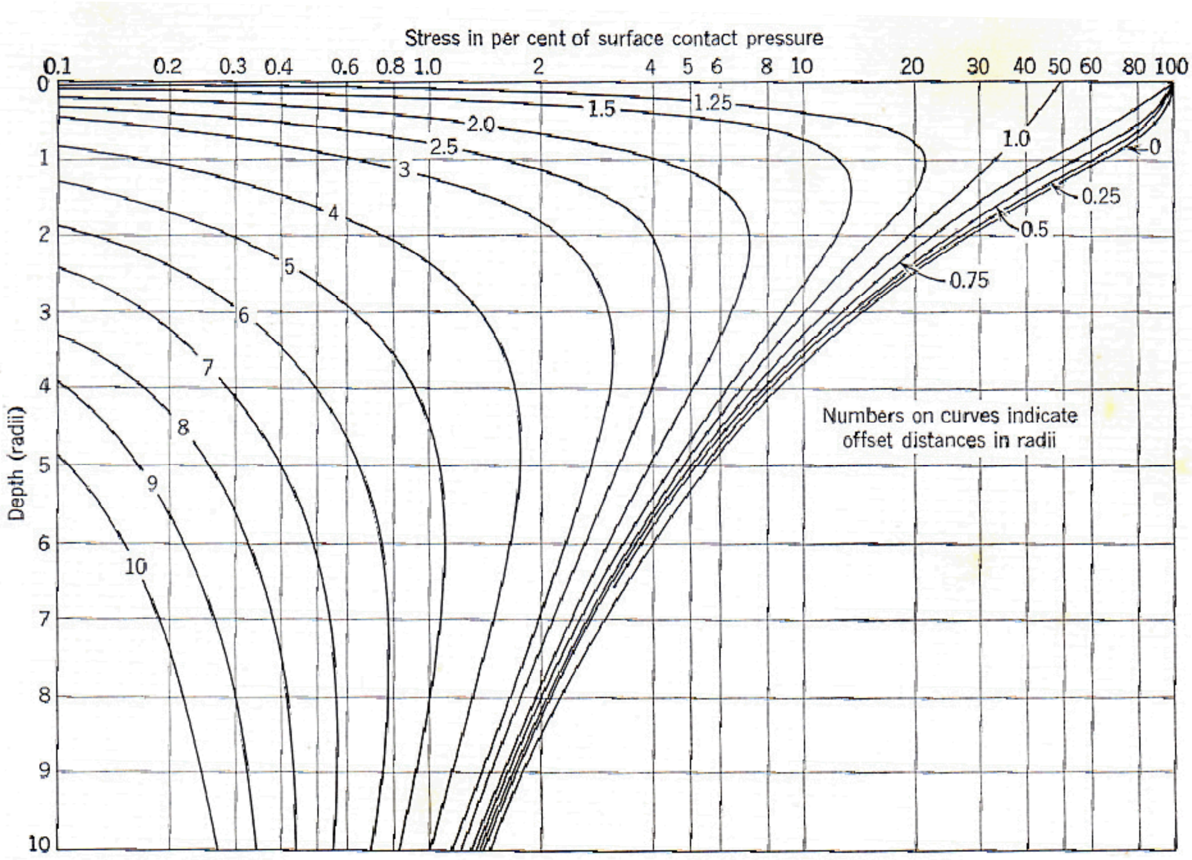


Figure 1.2 : Vertical Stress σ_z due to Circular Loading (Foster and Ahlvin, 1954)

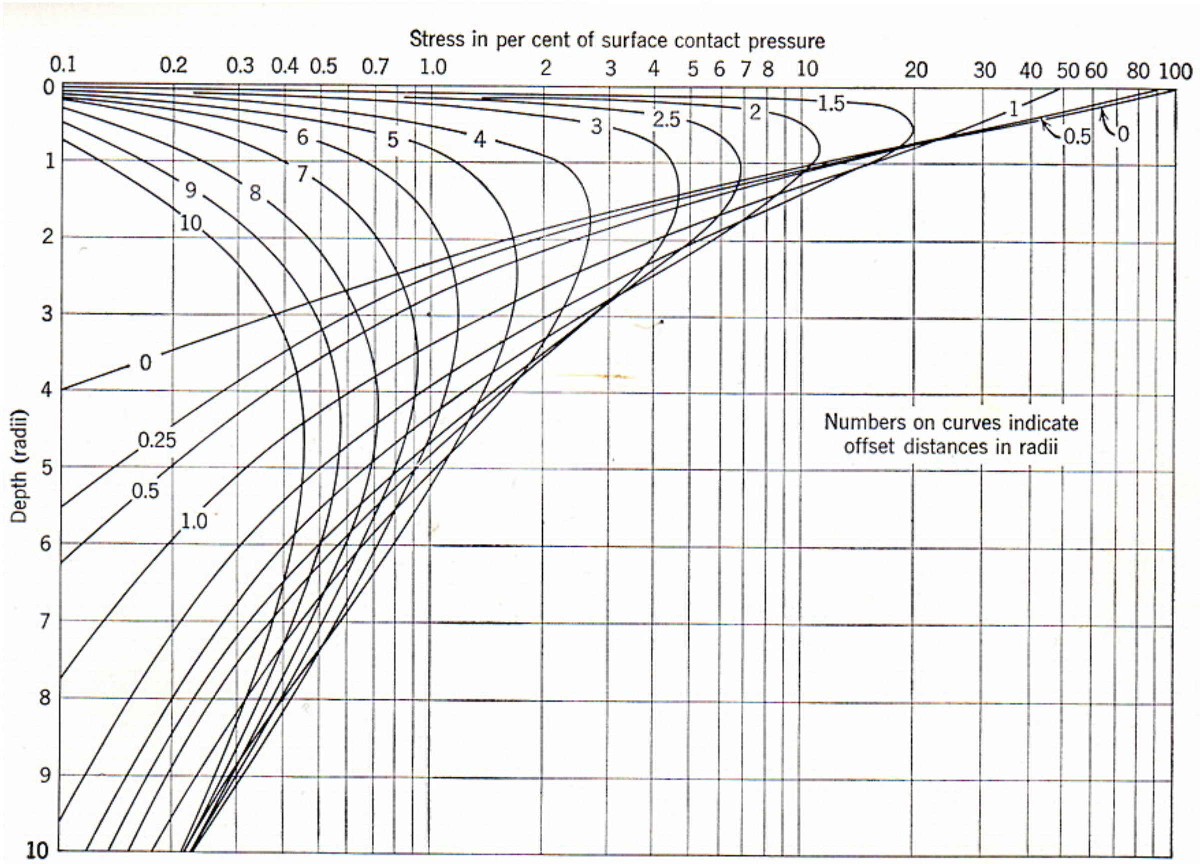


Figure 1.3 : Radial Stress σ_r due to Circular Loading (Foster and Ahlvin, 1954)

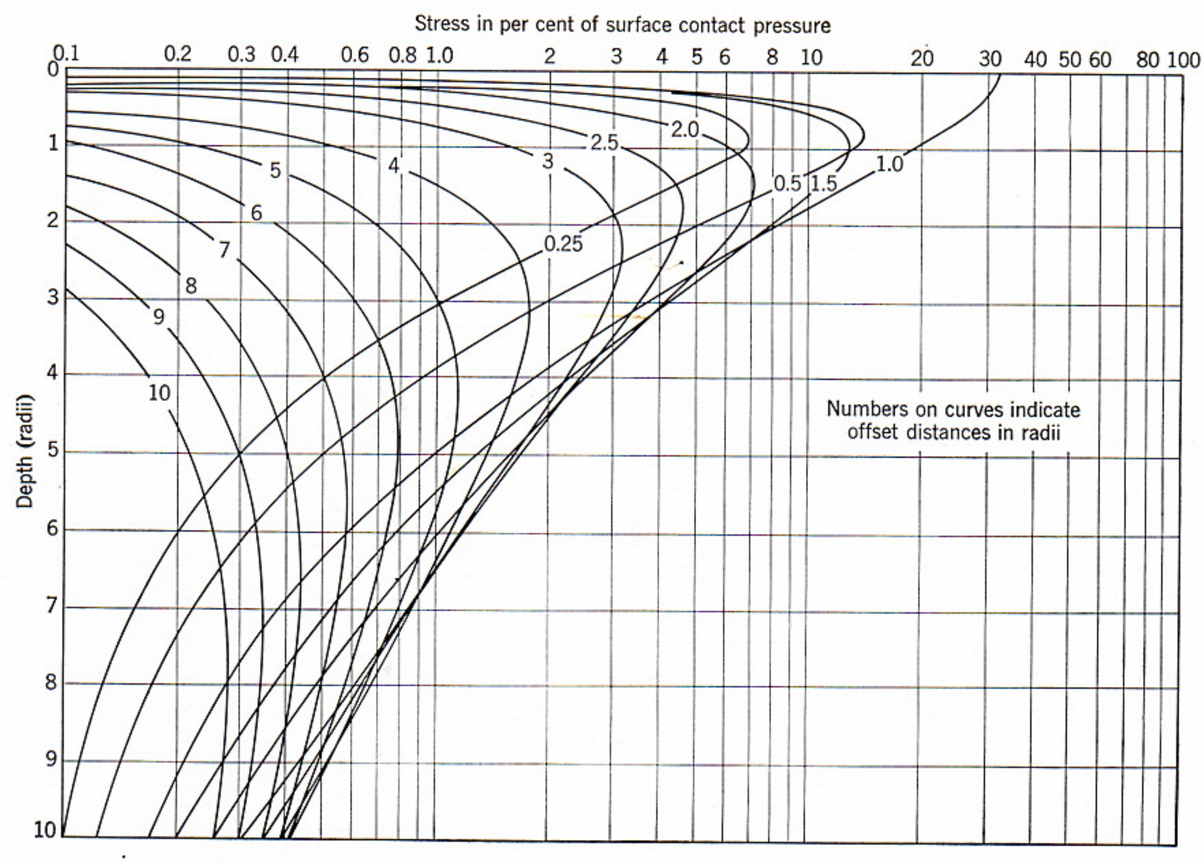


Figure 1.4 : Shear Stress τ_{zr} due to Circular Loading (Foster and Ahlvin, 1954)

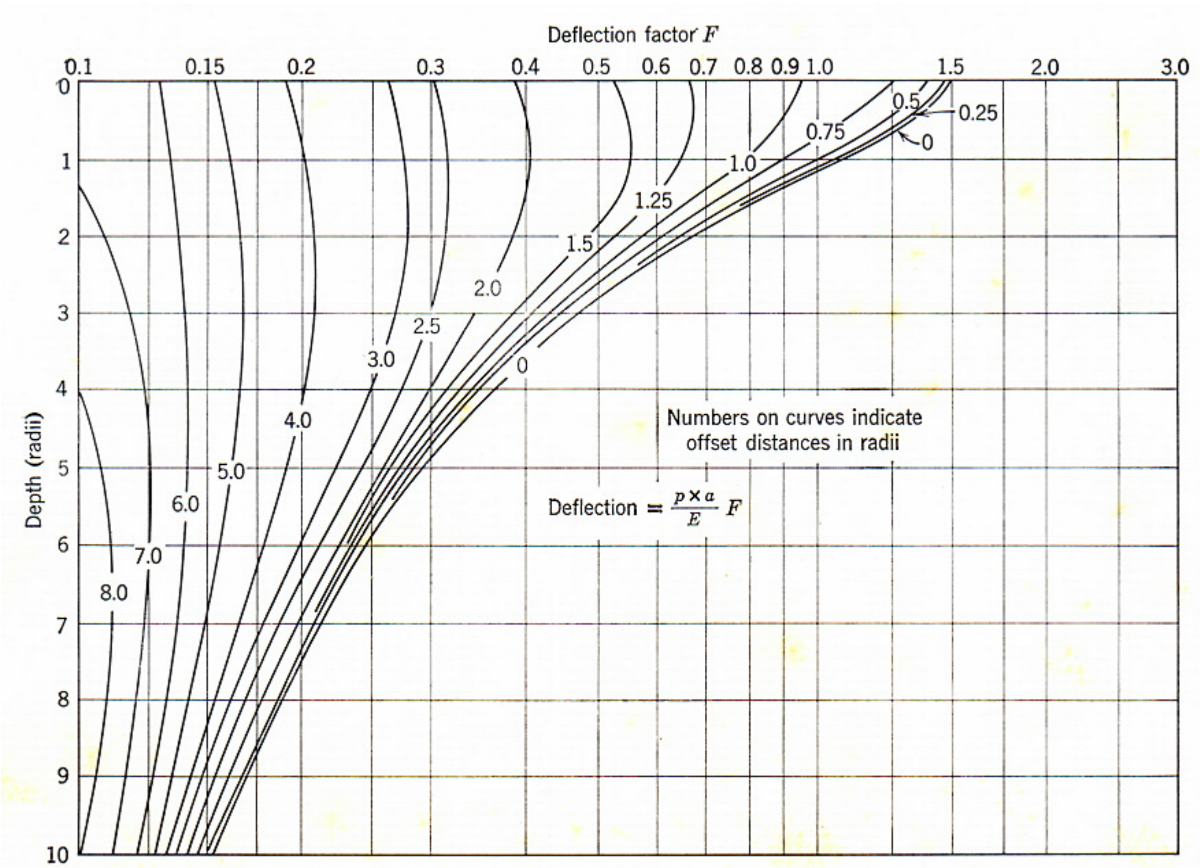


Figure 1.5 : Vertical Deflection w due to Circular Loading (Foster and Ahlvin, 1954)

1.3 STRESSES IN LAYERED SYSTEM

1.3.1 Two Layer Elastic Solutions

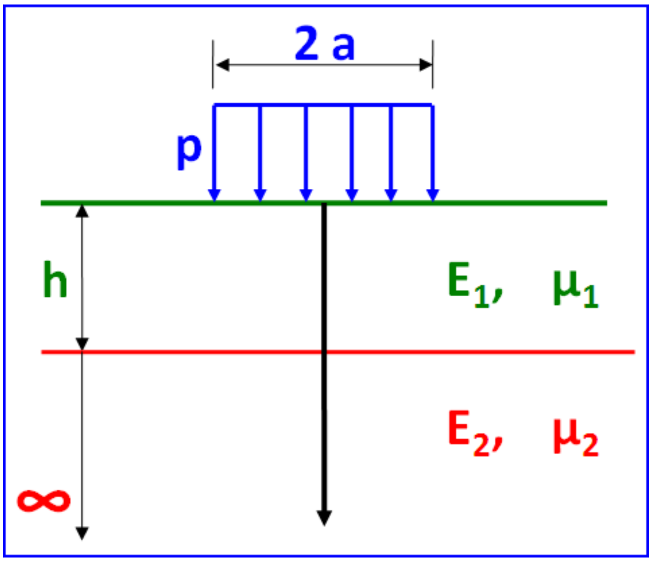


Figure 1.6 : General Two-Layer System

Vertical Stress Distribution

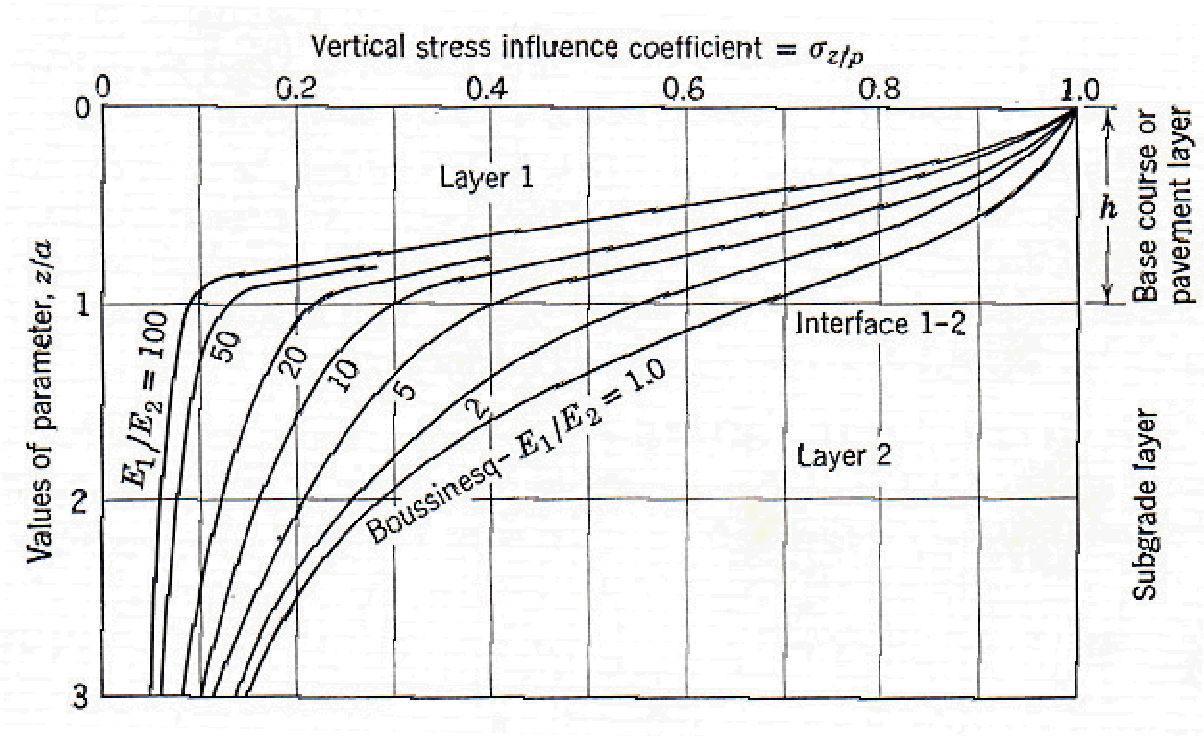


Figure 1.7 : Vertical Stress Distribution in Two-Layer Systems (Burmister, 1958)

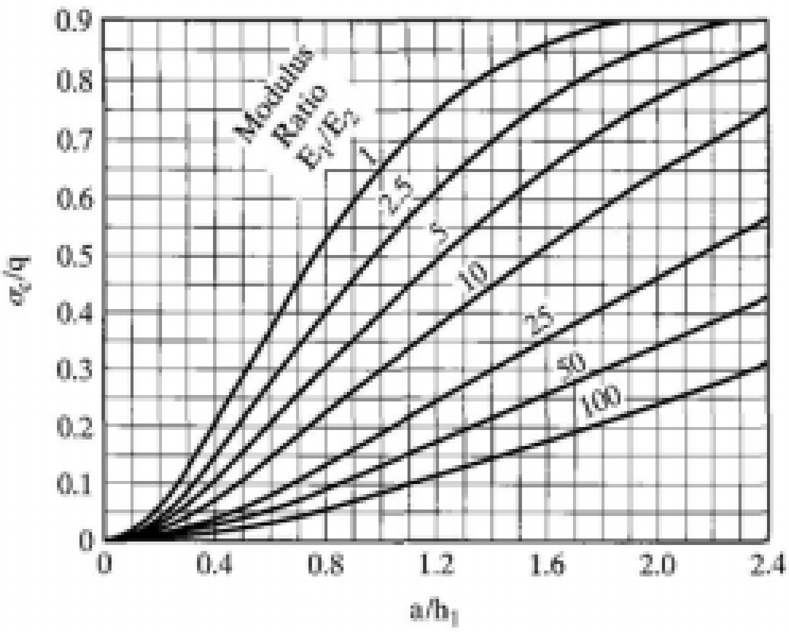


Figure 1.8 : Vertical Interface Stresses for Two-Layer Systems (Huang, 1969)

Vertical Surface Deflection

On a flexible plate $$w = 1.5\,\frac{q\,a}{E_2}F_2 \quad \text{when } \mu = 0.5$$ (1.7 a)

On a rigid plate $$w = 1.18\,\frac{q\,a}{E_2}F_2 \quad \text{when } \mu = 0.5$$ (1.7 b)

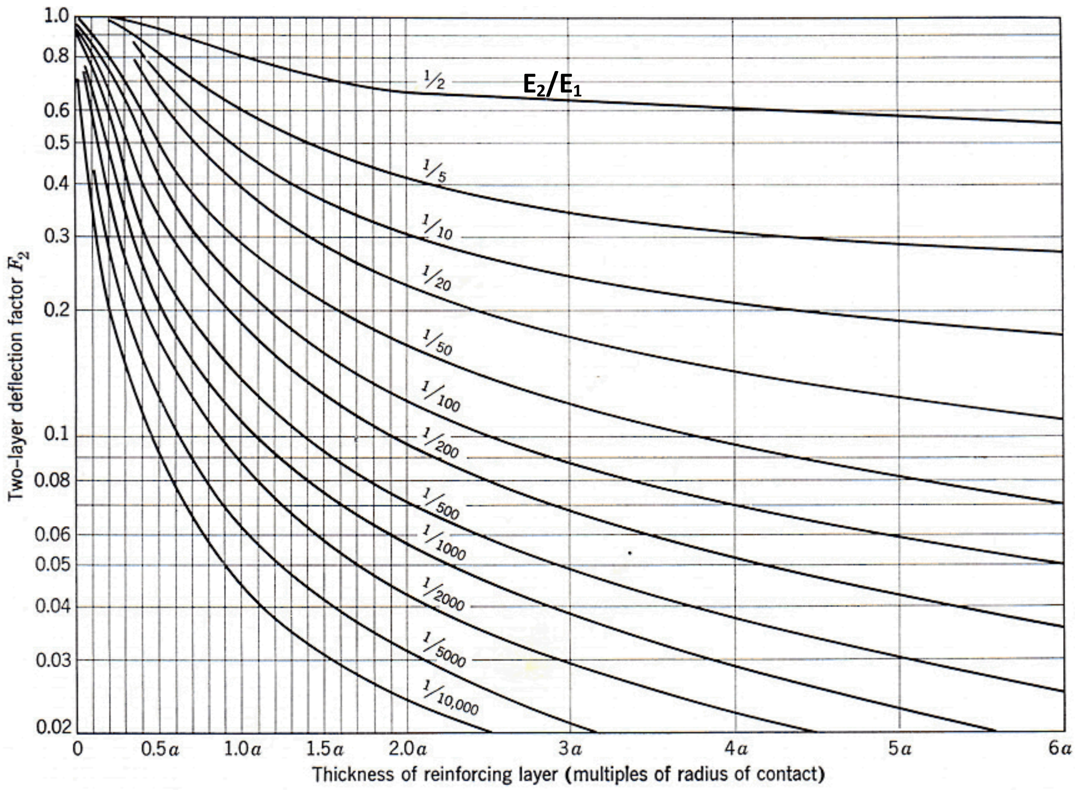


Figure 1.9 : Vertical Surface Deflections for Two-Layer System (Burmister, 1943)

Vertical Interface Deflection

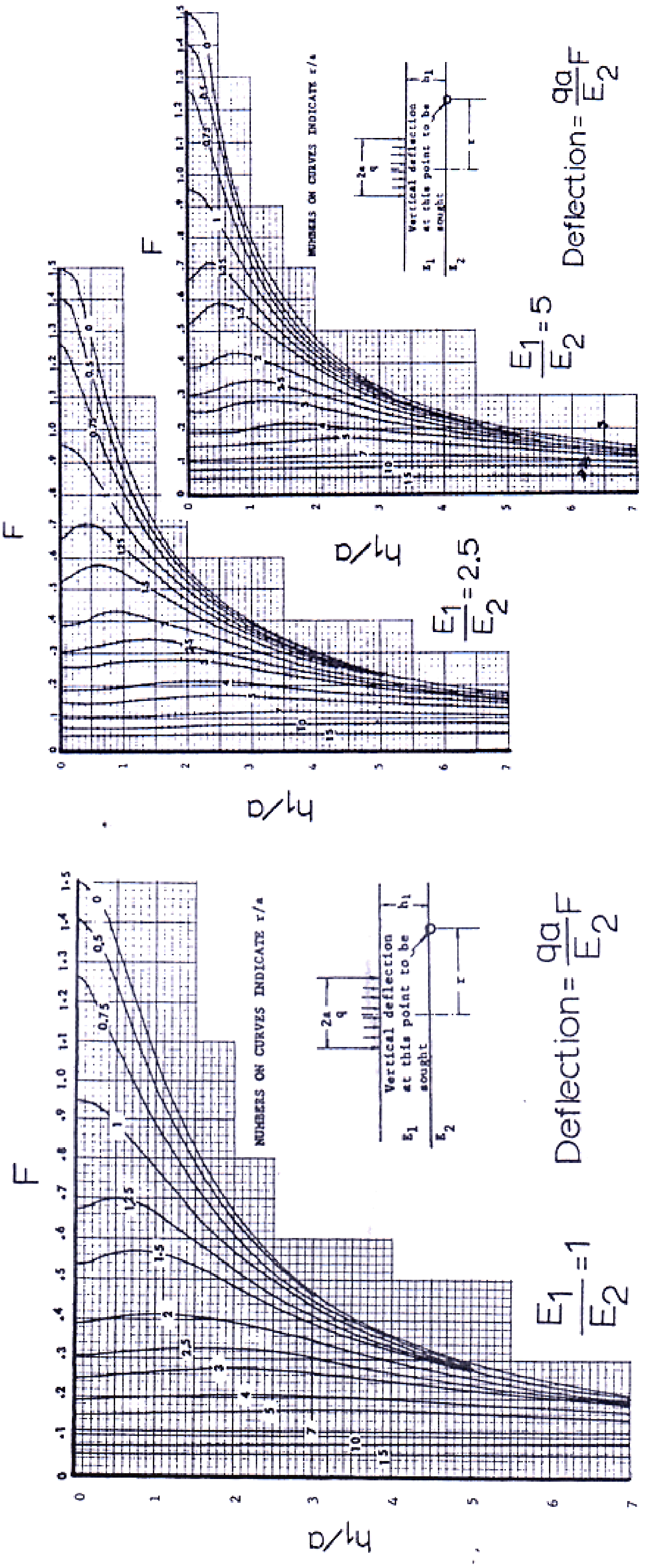


Figure 1.10 : Vertical Interface Deflection for Two-Layer Systems (Huang, 1969)

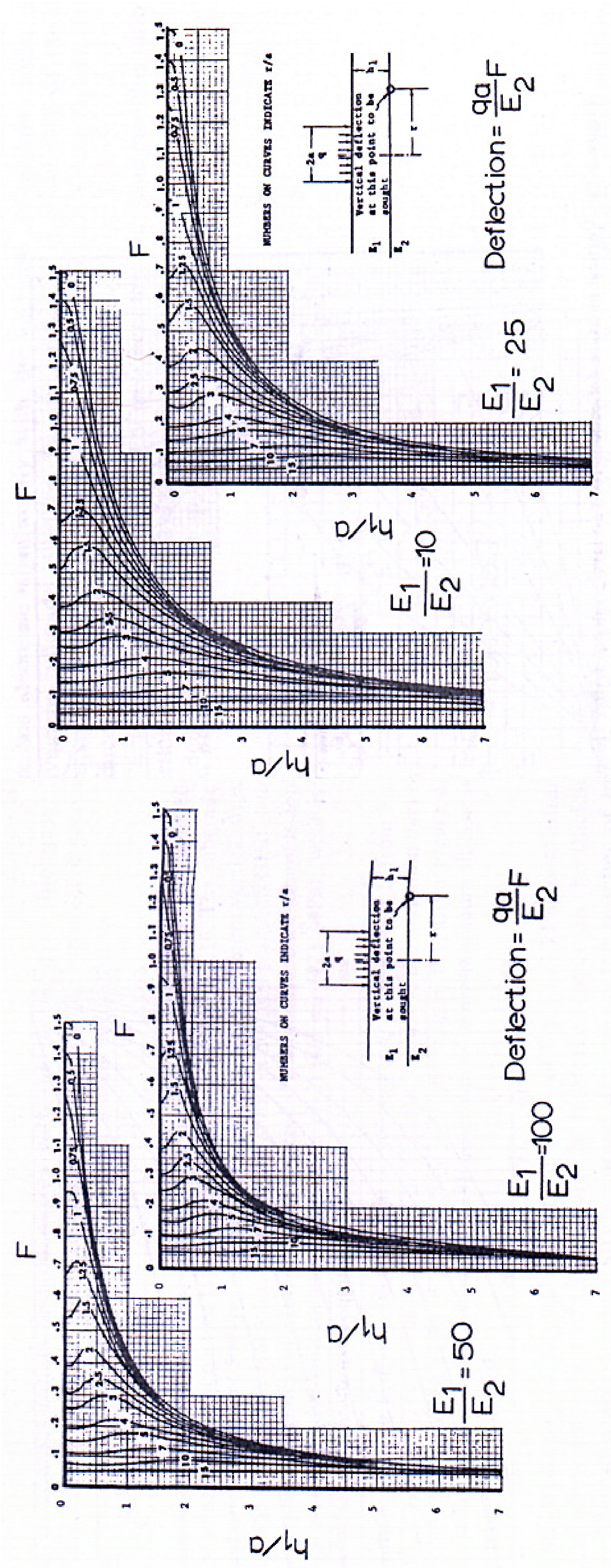


Figure 1.10 : Continued

1.3.2 Three Layer Elastic Solutions

Jones' Tables

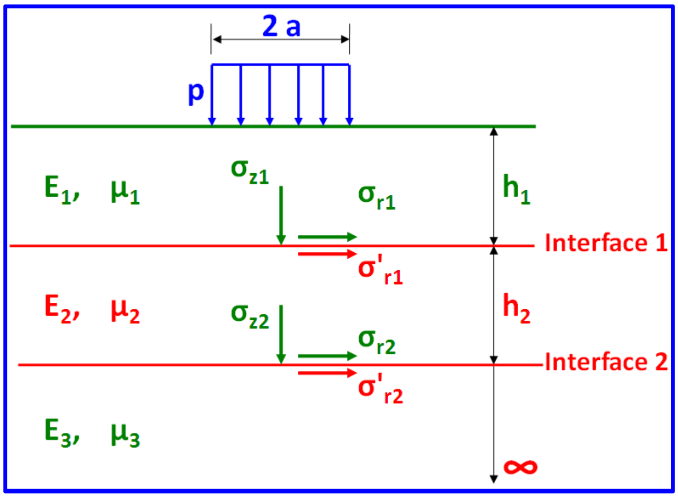


Figure 1.11 : General Three-Layer System

$$k_1 = \frac{E_1}{E_2} \qquad k_2 = \frac{E_2}{E_3} \qquad A = \frac{a}{h_2} \qquad H = \frac{h_1}{h_2} \tag{1.8 a}$$

$$\sigma_{z1} = p(ZZ1) \tag{1.8 b}$$

$$\sigma_{z2} = p(ZZ2) \tag{1.8 c}$$

$$\sigma_{z1} - \sigma_{r1} = p(ZZ1 - RR1) \tag{1.8 d}$$

$$\sigma_{z2} - \sigma_{r2} = p(ZZ2 - RR2) \tag{1.8 e}$$

$$\sigma_{z1} - \sigma'_{r1} = \frac{\sigma_{z1} - \sigma_{r1}}{k_1} \tag{1.8 f}$$

$$\sigma_{z2} - \sigma'_{r2} = \frac{\sigma_{z2} - \sigma_{r2}}{k_2} \tag{1.8 g}$$

Stress Factors for Three-Layer System (Jones, 1962)

H	K_2	A	$K_1 = 2$				$K_1 = 20$				$K_1 = 200$			
			ZZ1	ZZ2	ZZ1 – RR1	ZZ2 – RR2	ZZ1	ZZ2	ZZ1 – RR1	ZZ2 – RR2	ZZ1	ZZ2	ZZ1 – RR1	ZZ2 – RR2
0.125	2	0.1	0.42950	0.00896	0.70622	0.01716	0.14529	0.00810	1.81178	0.01542	0.03481	0.00549	3.02259	0.00969
		0.2	0.78424	0.03493	0.97956	0.06647	0.38799	0.03170	3.76886	0.06003	0.11491	0.02167	8.02452	0.03812
		0.4	0.98044	0.12667	0.70970	0.23531	0.78651	0.11650	5.16717	0.21640	0.33218	0.08229	17.64175	0.14286
		0.8	0.99434	0.36932	0.22319	0.63003	1.02218	0.34941	3.43631	0.60493	0.72695	0.27307	27.27701	0.45208
		1.6	0.99364	0.72113	-0.19982	0.97707	0.99060	0.69014	1.15211	0.97146	1.00203	0.63916	23.38638	0.90861
		3.2	0.99922	0.96148	-0.28916	0.84030	0.99893	0.93487	0.06894	0.88358	1.00828	0.92560	11.87014	0.91469
	20	0.1	0.43022	0.00228	0.69332	0.03467	0.14447	0.00182	1.80664	0.02985	0.03336	0.00128	3.17763	0.01980
		0.2	0.78414	0.00899	0.92086	0.13541	0.38469	0.00716	3.74573	0.11697	0.10928	0.00509	8.66097	0.07827
		0.4	0.97493	0.03392	0.46583	0.49523	0.77394	0.02710	5.05489	0.43263	0.31094	0.01972	20.12259	0.29887
		0.8	0.97806	0.11350	-0.66535	1.49612	0.98610	0.09061	2.92533	1.33736	0.65934	0.07045	36.29943	1.01694
		1.6	0.96921	0.31263	-2.82859	3.28512	0.93712	0.24528	-1.27093	2.99215	0.87931	0.20963	49.40857	2.64313
		3.2	0.98591	0.68433	-5.27906	5.05952	0.96330	0.55490	-7.35384	5.06489	0.93309	0.49938	57.84369	4.89895
0.25	2	0.1	0.15524	0.00710	0.28362	0.01353	0.04381	0.00530	0.63215	0.00962	0.00909	0.00259	0.96553	0.00407
		0.2	0.42809	0.02783	0.70225	0.05278	0.14282	0.02091	1.83766	0.03781	0.03269	0.01027	3.10763	0.01611
		0.4	0.77939	0.10306	0.96634	0.19178	0.37882	0.07933	3.86779	0.14159	0.10684	0.04000	8.37852	0.06221
		0.8	0.96703	0.31771	0.66885	0.55211	0.75904	0.26278	5.50796	0.44710	0.30477	0.14513	18.95534	0.21860
		1.6	0.98156	0.66753	0.17331	0.95080	0.98743	0.61673	4.24281	0.90115	0.66786	0.42940	31.18909	0.58553
		3.2	0.99840	0.93798	-0.05691	0.89390	1.00064	0.91258	1.97494	0.93254	0.98447	0.84545	28.98500	0.89191
	20	0.1	0.15436	0.00179	0.25780	0.02728	0.04236	0.00123	0.65003	0.01930	0.00776	0.00065	1.08738	0.00861
		0.2	0.42462	0.00706	0.67115	0.10710	0.13708	0.00488	1.90693	0.07623	0.02741	0.00257	3.59448	0.03421
		0.4	0.76647	0.02697	0.84462	0.39919	0.35716	0.01888	4.13976	0.29072	0.08634	0.01014	10.30923	0.13365
		0.8	0.92757	0.09285	0.21951	1.26565	0.68947	0.06741	6.48948	0.98565	0.23137	0.03844	26.41442	0.49135
		1.6	0.91393	0.26454	-1.22411	2.94860	0.85490	0.20115	6.95639	2.55231	0.46835	0.13148	57.46409	1.53833
		3.2	0.95243	0.60754	-3.04320	4.89878	0.90325	0.48647	6.05854	4.76234	0.71083	0.37342	99.29034	3.60964

Stress Factors for Three-Layer System (Jones, 1962)

H	K_2	A	$K_1 = 2$				$K_1 = 20$				$K_1 = 200$			
			ZZ1	**ZZ2**	**ZZ1 – RR1**	**ZZ2 – RR2**	**ZZ1**	**ZZ2**	**ZZ1 – RR1**	**ZZ2 – RR2**	**ZZ1**	**ZZ2**	**ZZ1 – RR1**	**ZZ2 – RR2**
0.5	2	0.1	0.04330	0.00465	0.08250	0.00878	0.01122	0.00259	0.17997	0.00440	0.00215	0.00094	0.26620	0.00128
		0.2	0.15325	0.01836	0.28318	0.03454	0.04172	0.01028	0.64779	0.01744	0.00826	0.00373	0.98772	0.00509
		0.4	0.42077	0.06974	0.70119	0.12954	0.13480	0.03998	1.89817	0.06722	0.02946	0.01474	3.19580	0.01996
		0.8	0.75683	0.23256	0.96681	0.41187	0.35175	0.14419	4.09592	0.23476	0.09508	0.05622	8.71973	0.07434
		1.6	0.93447	0.56298	0.70726	0.85930	0.70221	0.42106	6.22002	0.62046	0.27135	0.19358	20.15765	0.23838
		3.2	0.98801	0.88655	0.33878	0.96353	0.97420	0.82256	5.41828	0.93831	0.62399	0.52912	34.25229	0.54931
	20	0.1	0.04193	0.00117	0.08044	0.01778	0.00990	0.00063	0.19872	0.00911	0.00149	0.00023	0.31847	0.00257
		0.2	0.14808	0.00464	0.27574	0.07027	0.03648	0.00251	0.72264	0.03620	0.00564	0.00094	1.19598	0.01025
		0.4	0.40086	0.01799	0.67174	0.26817	0.11448	0.00988	2.19520	0.14116	0.01911	0.00372	1.02732	0.04047
		0.8	0.69098	0.06476	0.86191	0.91168	0.27934	0.03731	5.24726	0.51585	0.05574	0.01453	12.00885	0.15452
		1.6	0.79338	0.19803	0.39588	2.38377	0.50790	0.12654	10.30212	1.59341	0.13946	0.05399	32.77028	0.53836
		3.2	0.85940	0.49238	-0.41078	4.47022	0.70903	0.35807	16.38520	3.69109	0.30247	0.18091	77.62943	1.56409
1.0	2	0.1	0.01083	0.00241	0.02179	0.00453	0.00263	0.00100	0.04751	0.00160	0.00049	0.00029	0.06883	0.00035
		0.2	0.04176	0.00958	0.08337	0.01797	0.01029	0.00347	0.18481	0.00637	0.00195	0.00116	0.26966	0.00138
		0.4	0.14665	0.03724	0.28491	0.06934	0.03810	0.01565	0.66727	0.02498	0.00746	0.00460	1.00131	0.00545
		0.8	0.39942	0.13401	0.71341	0.24250	0.12173	0.05938	1.97428	0.09268	0.02647	0.01797	3.24971	0.02092
		1.6	0.71032	0.38690	1.02680	0.63631	0.31575	0.20098	4.37407	0.29253	0.08556	0.06671	8.92442	0.07335
		3.2	0.92112	0.75805	0.90482	0.97509	0.66041	0.53398	6.97695	0.65446	0.25186	0.22047	20.83387	0.21288
	20	0.1	0.00963	0.00061	0.02249	0.00920	0.00193	0.00024	0.05737	0.00322	0.00027	0.00007	0.08469	0.00062
		0.2	0.03697	0.00241	0.08618	0.03654	0.00751	0.00098	0.22418	0.01283	0.00104	0.00028	0.33312	0.00248
		0.4	0.12805	0.00950	0.29640	0.14241	0.02713	0.00387	0.82430	0.05063	0.00384	0.00110	1.25495	0.00985
		0.8	0.33263	0.03578	0.76292	0.51815	0.08027	0.01507	2.59672	0.19267	0.01236	0.00436	4.26100	0.03825
		1.6	0.52721	0.12007	1.25168	1.56503	0.17961	0.05549	6.77014	0.66326	0.03379	0.01683	12.91809	0.13989
		3.2	0.65530	0.33669	1.70723	3.51128	0.34355	0.18344	15.23252	1.88634	0.08859	0.06167	36.04291	0.45544

Stress Factors for Three-Layer System (Jones, 1962)

H	K_2	A	$K_1 = 2$				$K_1 = 20$				$K_1 = 200$			
			ZZ1	ZZ2	ZZ1 – RR1	ZZ2 – RR2	ZZ1	ZZ2	ZZ1 – RR1	ZZ2 – RR2	ZZ1	ZZ2	ZZ1 – RR1	ZZ2 – RR2
2.0	2	0.1	0.00250	0.00100	0.00555	0.00188	0.00059	0.00033	0.01219	0.00051	0.00011	0.00008	0.01737	0.00009
		0.2	0.00991	0.00397	0.02199	0.00750	0.00235	0.00130	0.04843	0.00203	0.00045	0.00033	0.06913	0.00036
		0.4	0.03832	0.01569	0.08465	0.02950	0.00922	0.00518	0.18857	0.00803	0.00179	0.00131	0.27103	0.00142
		0.8	0.13516	0.05974	0.29365	0.11080	0.03412	0.02023	0.68382	0.03093	0.00685	0.00520	1.00808	0.00553
		1.6	0.36644	0.20145	0.75087	0.35515	0.10918	0.07444	2.04134	0.10864	0.02441	0.02003	3.27590	0.02043
		3.2	0.67384	0.51156	1.17294	0.77434	0.29183	0.23852	4.60426	0.30709	0.08061	0.07248	9.02195	0.06638
	20	0.1	0.00181	0.00025	0.00652	0.00378	0.00033	0.00008	0.01568	0.00094	0.00005	0.00002	0.02160	0.00014
		0.2	0.00716	0.00099	0.02586	0.01507	0.00130	0.00031	0.06236	0.00374	0.00018	0.00007	0.08604	0.00058
		0.4	0.02746	0.00394	0.10017	0.05958	0.00503	0.00123	0.24425	0.01486	0.00071	0.00030	0.33866	0.00229
		0.8	0.09396	0.01535	0.35641	0.22795	0.01782	0.00485	0.90594	0.05789	0.00261	0.00119	1.27835	0.00901
		1.6	0.23065	0.05599	1.00785	0.78347	0.05012	0.01862	2.91994	0.21190	0.00819	0.00467	4.35311	0.03390
		3.2	0.37001	0.17843	2.16033	2.13215	0.11331	0.06728	7.95104	0.67732	0.02341	0.01784	13.26873	0.11666
4.0	2	0.1	0.00057	0.00034	0.00147	0.00065	0.00013	0.00010	0.00312	0.00015	0.00003	0.00002	0.00437	0.00002
		0.2	0.00228	0.00137	0.00587	0.00260	0.00054	0.00039	0.01245	0.00029	0.00011	0.00009	0.01746	0.00009
		0.4	0.00905	0.00544	0.02324	0.01032	0.00214	0.00154	0.04944	0.00235	0.00042	0.00036	0.06947	0.00036
		0.8	0.03500	0.02135	0.08957	0.04031	0.00837	0.00610	0.19247	0.00924	0.00168	0.00142	0.27221	0.00144
		1.6	0.12354	0.07972	0.31215	0.14735	0.03109	0.02358	0.69749	0.03488	0.00646	0.00560	1.01140	0.00553
		3.2	0.34121	0.25441	0.81908	0.43632	0.10140	0.08444	2.09049	0.11553	0.02332	0.02126	3.28913	0.01951
	20	0.1	0.00030	0.00008	0.00201	0.00128	0.00005	0.00002	0.00413	0.00025	0.00001	0.00000	0.00545	0.00003
		0.2	0.00119	0.00034	0.00803	0.00510	0.00021	0.00009	0.01651	0.00099	0.00003	0.00002	0.02178	0.00014
		0.4	0.00469	0.00134	0.03191	0.02032	0.00083	0.00035	0.06569	0.00396	0.00013	0.00008	0.08673	0.00054
		0.8	0.01790	0.00532	0.12427	0.07991	0.00321	0.00138	0.25739	0.01565	0.00050	0.00031	0.34131	0.00215
		1.6	0.06045	0.02049	0.45100	0.29991	0.01103	0.00542	0.95622	0.05993	0.00186	0.00124	1.28773	0.00833
		3.2	0.14979	0.07294	1.36427	0.97701	0.03258	0.02061	3.10980	0.20906	0.00612	0.00483	4.38374	0.03010

1.4 DESIGN OF FLEXIBLE PAVEMENTS (IRC : 37-2001)

1.4.1 Design Traffic

The design traffic is considered in terms of cumulative number of standard axles (in the lane carrying maximum traffic) to be carried during the design life of pavement using

$$N = \frac{365[(1+r)^n - 1]}{r} * A * D * F \qquad (1.9\ a)$$

N	The cumulative number of standard axles to be catered for in the design life in terms of msa
A	Initial traffic in the year of completion of construction in terms of the number of commercial vehicles per day
D	Lane distribution factor
F	Vehicle damage factor
n	Design life in years
r	Annual growth rate of commercial vehicles

The traffic in the year of completion is estimated using

$$A = P(1+r)^x \qquad (1.9\ b)$$

P	Number of commercial vehicles as per last count
x	Number of years between the last count and the year of completion of construction

1.4.2 Traffic growth rate

Traffic growth rates should be estimated

by studying the past trends of traffic growth, and

by establishing econometric models, as per the procedure outlined in IRC:108 “Guidelines for traffic prediction on rural highways”.

If adequate data is not available, it is recommended that an average annual growth rate of 7.5 percent may be adopted.

1.4.3 Design Life

For the design of pavement, the design life is defined in terms of the cumulative number of standard axles that can be carried before strengthening of pavement is necessary.

It is recommended that pavements for National Highways (NH) and State Highways (SH) should be design for a life of 15 years. Expressways and Urban roads nay be designed for a longer life of 20 years. For other categories of roads, a design life of 10 to 15 years may be adopted.

1.4.4 Vehicle Damage Factor

$$VDF = \frac{V_1\left(\frac{W_1}{W_s}\right)^4 + V_2\left(\frac{W_2}{W_s}\right)^4 + V_3\left(\frac{W_3}{W_s}\right)^4 + \dots\dots}{V_1 + V_2 + V_3 + \dots\dots} \quad (1.10\text{ a})$$

$$VDF = \frac{V_1 EALF_1 + V_2 EALF_2 + V_3 EALF_3 + \dots\dots}{V_1 + V_2 + V_3 + \dots\dots} \quad (1.10\text{ b})$$

$$EALF = \left(\frac{Axle\ Load}{Standard\ Axle\ Load}\right)^4 \quad (1.10\text{ c})$$

Standard Axle Load

Single Axle : 8160 kg

Tandem Axle : 14968 kg

Where sufficient information on axle loads is not available and project does not warrant conducting an axle load survey, the indicative values of vehicle damage factor as given below may be used.

Indicative VDF Values (Table 1 of IRC:37-2001)

Initial traffic volume (CVPD)	Terrain	
	Rolling/Plain	Hilly
0-150	1.5	0.5
150-1500	3.5	1.5
More than 1500	4.5	2.5

1.4.5 Distribution of Commercial traffic over the carriageway

In the absence of adequate and conclusive data for Indian conditions, it is recommended to assume the following distribution.

No. of Traffic lanes in two directions	**Percentage of trucks in Design Lane**	
	Undivided Roads (Single Carriageway)	**Divided Roads (Dual Carriageway)**
1	100	100
2	75	75
3	----	60
4	40	45

1.4.6 Design Criteria

The flexible pavements has been modeled as a three layer structure and stresses and strains at critical locations have been computed using the linear elastic model. To consider the aspects of performance, the following three types of pavement distress resulting from repeated (cyclic) application of traffic loads are considered:

- Vertical compressive strain at the top of the sub-grade which can cause sub-grade deformation resulting in permanent deformation at the pavement surface.
- Horizontal tensile strain or stress at the bottom of the bituminous layer which can cause fracture of the bituminous layer.
- Pavement deformation within the bituminous layer.

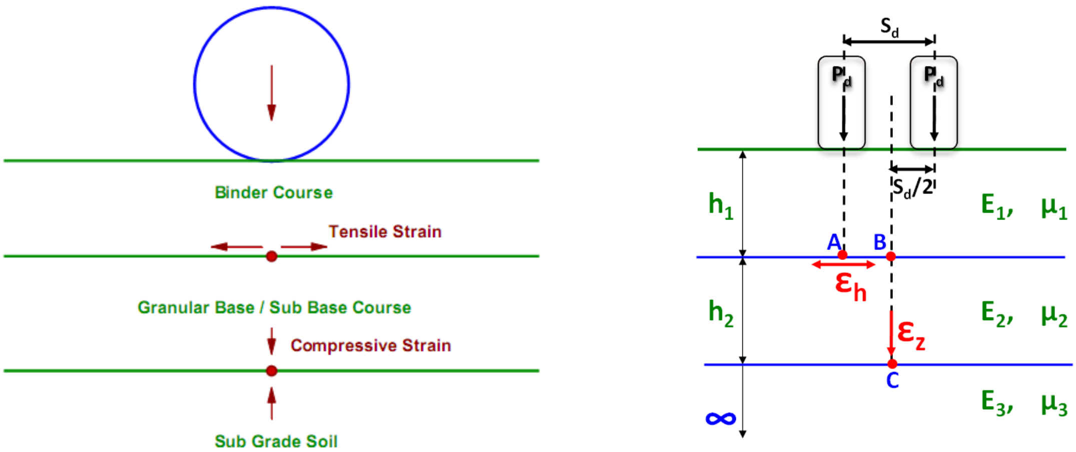


Figure 1.11 : Critical Locations in Pavement

While the permanent deformation within the bituminous layer can be controlled by meeting the mix design requirements, thickness of granular and bituminous layers are selected using the analytical design approach so that strains at the critical points are within the allowable limits. For calculating tensile strains at the bottom of the bituminous layer, the stiffness of dense bituminous macadam (DBM) layer with 60/70 bitumen has been used in the analysis.

1.4.7 Failure Criteria

As shown in figure 2.11, A and B are the critical locations for tensile strains (ε_t). Maximum value of the strain is adopted for design. C is the critical location for the vertical subgrade strain (ε_z) since the maximum value of the ε_z occurs mostly at C.

Fatigue Criteria:

Bituminous surfacing of pavements display flexural fatigue cracking if the tensile strain at the bottom of the bituminous layer is beyond certain limit. The relation between the fatigue life of the pavement and the tensile strain in the bottom of the bituminous layer is expressed as

$$N_f = 2.21 \times 10^{-4} \left(\frac{1}{\varepsilon_t}\right)^{3.89} \left(\frac{1}{E}\right)^{0.854} \tag{1.11}$$

N_f Allowable number of load repetitions to produce 20% cracked surface area
ε_t Tensile strain at the bottom of surface layer (micro strain)
E Elastic modulus of bituminous surfacing (MPa)

Rutting Criteria:

The allowable number of load repetitions to control permanent deformation can be expressed as

$$N_r = 4.1656 \times 10^{-8} \left(\frac{1}{\varepsilon_z}\right)^{4.5337} \tag{1.12}$$

N_r Allowable number of load repetitions to produce rutting of 20 mm
ε_z Vertical subgrade strain (micro strain)

Standard axle load considered is 80 kN. One dual wheel set with a wheel load of 20kN, center-to-center tyre spacing of 310 mm and tyre pressure of 0.56 MPa is considered for analysis.

1.4.8 Design Charts and Catalogue

Based on the performance of existing designs and using analytical approach, simple design charts (Figure 2.13 and 2.14) and a catalogue of pavement designs are added in the code. The pavement designs are given for subgrade CBR values ranging from 2% to 10% and design traffic ranging from 1 msa to 150 msa for an average annual pavement temperature of 35 C. The later thicknesses obtained from the analysis have been slightly modified to adapt the designs to stage construction. Using the following simple input parameters, appropriate designs could be chosen for the given traffic and soil strength:

- Design traffic in terms of cumulative number of standard axles; and
- CBR value of subgrade.

The designs relate to ten levels of design traffic 1, 2, 3, 4, 5, 10, 20, 30, 50, 100 and 150 msa. For intermediate traffic ranges, the pavement layer thickness may be interpolated linearly. For traffic exceeding 150 msa, the pavement design appropriate to 150 msa may be chosen and further strengthening carried out to extend the life at appropriate time based on pavement deflection measurements as per IRC : 81.

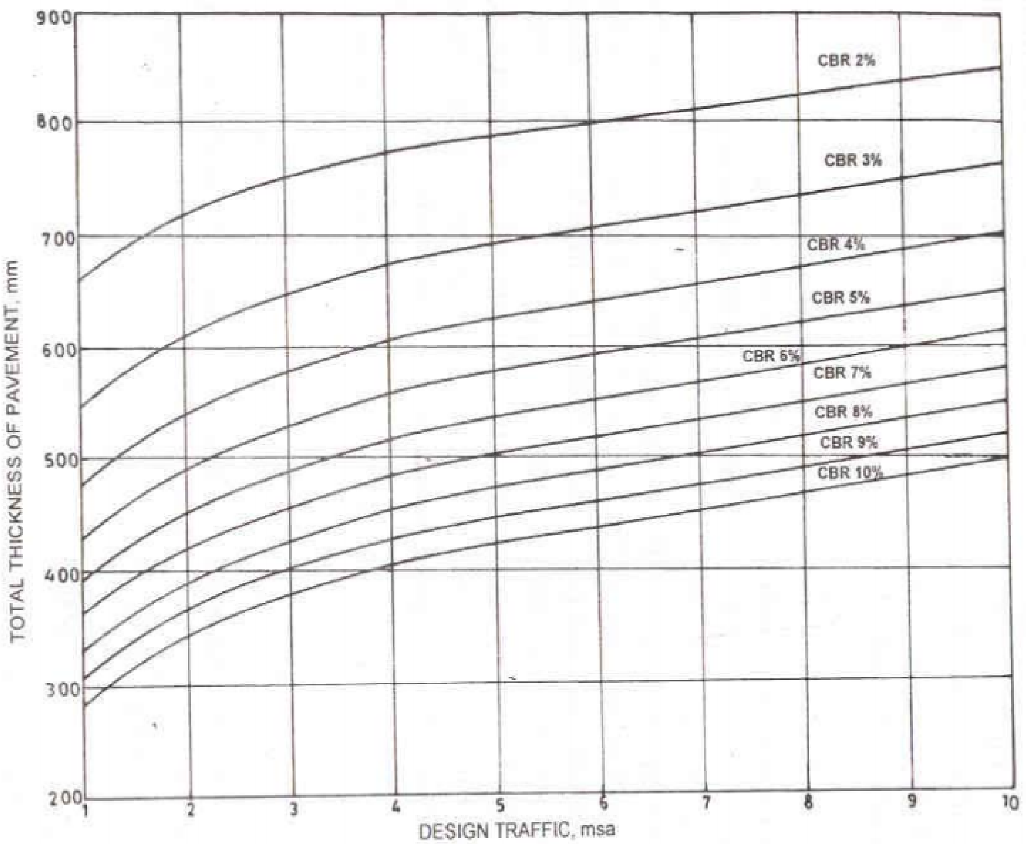


Figure 1.13 : Pavement Thickness Design Chart for Traffic 1-10 msa

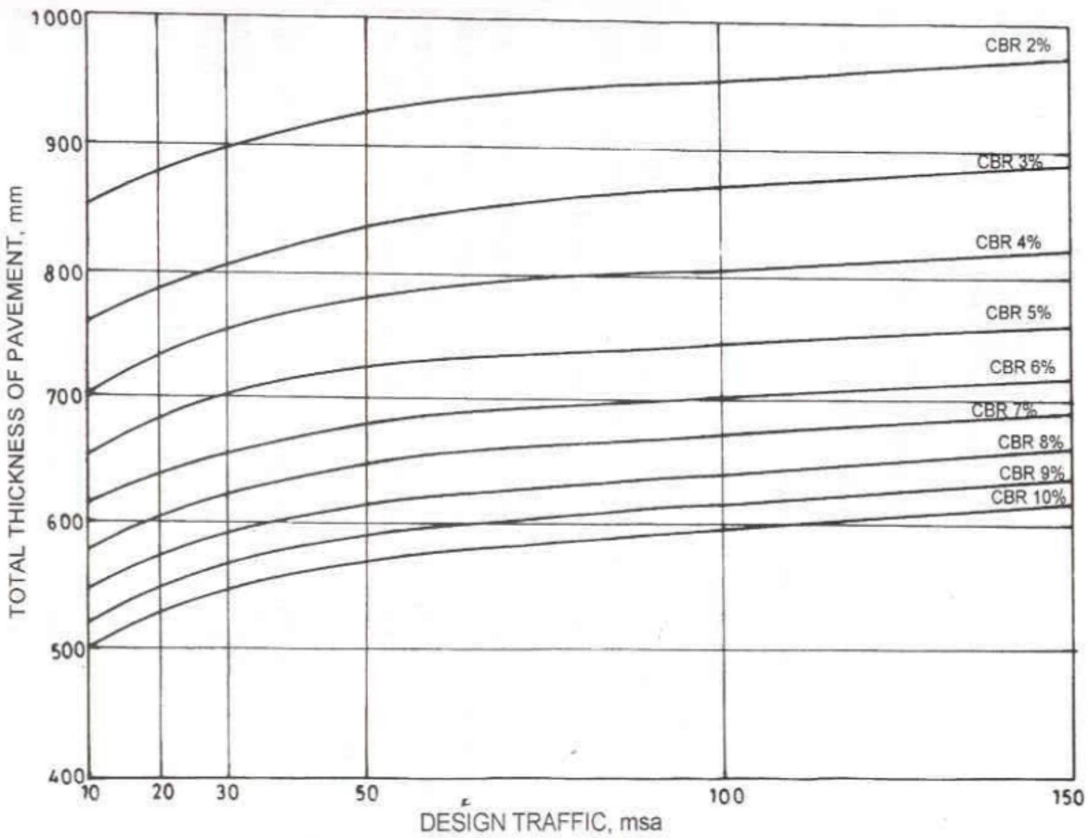


Figure 1.14 : Pavement Thickness Design Chart for Traffic 10-150 msa

Pavement Design Catalogue					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION (mm)			
		Bituminous Surfacing		Granular Base	Granular Sub-base
		Wearing Course	Binder Course		
CBR 2 %					
1	660	20 PC	------	225	435
2	715	20 PC	50 BM	225	440
3	750	20 PC	60 BM	250	440
5	795	25 SDBC	70 DBM	250	450
10	850	40 BC	100 DBM	250	460
20	880	40 BC	130 DBM		
30	900	40 BC	150 DBM		
50	925	40 BC	175 DBM		
100	955	40 BC	195 DBM		
150	975	50 BC	215 DBM		
CBR 3 %					
1	550	20 PC	------	225	435
2	610	20 PC	50 BM	225	335
3	645	20 PC	60 BM	250	335
5	690	25 SDBC	60 DBM	250	335
10	760	40 BC	90 DBM	250	380
20	790	40 BC	120 DBM		
30	810	40 BC	140 DBM		
50	830	40 BC	160 DBM		
100	860	50 BC	180 DBM		
150	890	50 BC	210 DBM		
CBR 4%					
1	480	20 PC	------	225	255
2	540	20 PC	50 BM	225	265
3	580	20 PC	50 BM	250	280
5	620	25 SDBC	60 DBM	250	285
10	700	40 BC	80 DBM	250	330
20	730	40 BC	110 DBM		
30	750	40 BC	130 DBM		
50	780	40 BC	160 DBM		
100	800	50 BC	170 DBM		
150	820	50 BC	190 DBM		

Pavement Design Catalogue

Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION (mm)			
		Bituminous Surfacing		Granular Base	Granular Sub-base
		Wearing Course	Binder Course		
CBR 5%					
1	430	20 PC	------	225	205
2	490	20 PC	50 BM	225	215
3	530	20 PC	50 BM	250	230
5	580	25 SDBC	55 DBM	250	250
10	660	40 BC	70 DBM	250	300
20	690	40 BC	100 DBM		
30	710	40 BC	120 DBM		
50	730	40 BC	140 DBM		
100	750	50 BC	150 DBM		
150	770	50 BC	170 DBM		
CBR 6 %					
1	390	20 PC	------	225	165
2	450	20 PC	50 BM	225	175
3	490	20 PC	50 BM	250	190
5	535	25 SDBC	50 DBM	250	210
10	615	40 BC	65 DBM	250	260
20	640	40 BC	90 DBM		
30	655	40 BC	105 DBM		
50	675	40 BC	125 DBM		
100	700	50 BC	140 DBM		
150	720	50 BC	160 DBM		
CBR 7%					
1	375	20 PC	------	225	150
2	425	20 PC	50 BM	225	150
3	460	20 PC	50 BM	250	160
5	505	25 SDBC	50 DBM	250	180
10	580	40 BC	60 DBM	250	230
20	610	40 BC	90 DBM		
30	630	40 BC	110 DBM		
50	650	40 BC	130 DBM		
100	675	50 BC	145 DBM		
150	695	50 BC	165 DBM		

Pavement Design Catalogue					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION (mm)			
		Bituminous Surfacing		Granular Base	Granular Sub-base
		Wearing Course	Binder Course		
CBR 8%					
1	375	20 PC	------	225	150
2	425	20 PC	50 BM	225	150
3	450	20 PC	50 BM	250	150
5	475	25 SDBC	50 DBM	250	150
10	550	40 BC	60 DBM	250	200
20	575	40 BC	85 DBM		
30	590	40 BC	100 DBM		
50	610	40 BC	120 DBM		
100	640	50 BC	140 DBM		
150	660	50 BC	160 DBM		
CBR 9%					
1	375	20 PC	------	225	150
2	425	20 PC	50 BM	225	150
3	450	20 PC	50 BM	250	150
5	475	25 SDBC	50 DBM	250	150
10	540	40 BC	50 DBM	250	200
20	570	40 BC	80 DBM		
30	585	40 BC	95 DBM		
50	605	40 BC	115 DBM		
100	635	50 BC	135 DBM		
150	655	50 BC	155 DBM		
CBR 10 %					
1	375	20 PC	------	225	150
2	425	20 PC	50 BM	225	150
3	450	20 PC	50 BM	250	150
5	475	25 SDBC	50 DBM	250	150
10	540	40 BC	50 DBM	250	200
20	565	40 BC	75 DBM		
30	580	40 BC	90 DBM		
50	600	40 BC	110 DBM		
100	630	50 BC	130 DBM		
150	650	50 BC	150 DBM		

1.4.9 Pavement Composition

Sub-base Course

- Natural sand, gravel, laterite, brick metal, crushed stone or combinations thereof
- Minimum CBR :
  - 20% upto 2 msa traffic
  - 30% exceeding 2 msa
- Minimum Thickness
  - 150 mm for traffic < 10 msa
  - 200 mm for traffic ≥ 10 msa
- If subgrade CBR < 2%, design for subgrade CBR of 2% and provide a 150 mm thick capping layer of minimum CBR 10% in addition to sub-base

Base Course

- Unbound granular material – WBM, WMM or other equivalent granular construction conforming to IRC/MORT&H specifications
- Minimum Thickness
  - 225 mm for traffic ≤ 2 msa
  - 250 mm for traffic > 2 msa
- If WBM is used and traffic > 10 msa, minimum thickness is 300 mm (4 layers of 75 mm each)

Bituminous Surfacing

- Wearing course or Binder course+wearing course
- Wearing course : Surface dressing, open-graded premix carpet, mix seal surfacing, SDBC and BC
- Binder course : BM, DBM, mix seal surfacing, SDBC and BC
- Wearing surface used is open-graded premix carpet of thickness upto 25 mm, it should not be counted towards the total thickness

1.4.10 Final Remarks

- The present guidelines follows mechanistic empirical approach and developed new set of designs up to 150 msa
- Thickness charts are still available for CBR values of up to 10% only
- Design charts are available for only a pavement temperature of 35^{o} C
- The contribution of individual component layers is still not realized fully with the system of catalogue thicknesses. The same can be done with the analytical tool for design.

2. ANALYSIS AND DESIGN OF RIGID PAVEMENTS

2.1 Modulus of Subgrade Reaction (K)

$$K = \frac{p}{\Delta} \tag{2.1 a}$$

p Pressure sustained by a rigid plate of diameter 75 cm at design deflection Δ
Δ Design deflection = 0.125 cm

❖ **Allowance for Worst Subgrade Moisture**

$$K_s = K\frac{p_s}{p_{us}} \tag{2.1 b}$$

p_{us} Pressure required in the plate bearing test for design deflection of 0.125 cm which produces a deformation of δ in unsoaked consolidation test
p_s Pressure required to produce the same deformation δ in the soaked consolidation test
K Modulus of subgrade reaction for the prevailing moisture condition
K_s Corrected modulus of subgrade reaction for worst subgrade moisture

❖ **Correction for Small Plate Size**

$$K = K_1\frac{a_1}{a} \tag{2.1 c}$$

K_1 Modulus of subgrade reaction determined using plate of radius a_1
K Corrected modulus of subgrade reaction for standard plate of radius a

2.2 Radius of Relative Stiffness (l)

$$l = \left[\frac{Eh^3}{12K(1-\mu^2)}\right]^{1/4} \tag{2.2}$$

E Modulus of elasticity of cement concrete
μ Poisson's ratio of concrete = 0.15
h Slab thickness
K Modulus of subgrade reaction

2.3 Equivalent Radius of Resisting Section (b)

$$b = \sqrt{1.6a^2 + h^2} - 0.675\,h \quad when\ a < 1.724\,h \tag{2.3}$$

$$b = a \qquad when\ a \geq 1.724\,h$$

A Radius of wheel load distribution
H Slab thickness

2.4 Critical Load Positions

The three typical locations namely the interior, edge and corner, where differing conditions of slab continuity exist, are treated as critical load positions.

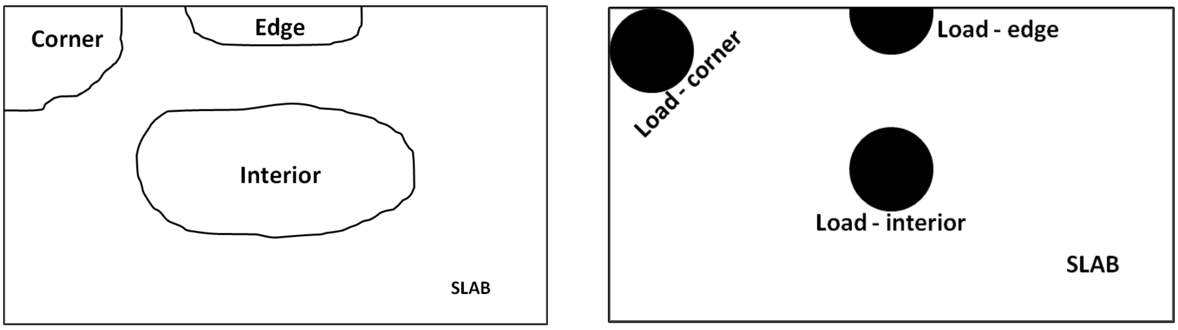


Figure 3.1: Critical Load Positions

2.5 Stresses and Deflections due to Wheel Load

2.5.1 <u>Corner Loading</u>

<u>Westergaard (1926)</u>

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right] \quad (2.3\ a)$$

$$\Delta_c = \frac{P}{Kl^2}\left[1.1 - 0.88\left(\frac{a\sqrt{2}}{l}\right)\right] \quad (2.3\ b)$$

<u>Westergaard analysis modified by Kelly</u>

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \quad (2.3\ c)$$

<u>Ioannides et al (1985)</u>

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{c}{l}\right)^{0.72}\right] \quad (2.3\ d)$$

$$\Delta_c = \frac{P}{Kl^2}\left[1.205 - 0.69\left(\frac{c}{l}\right)\right] \quad (2.3\ e)$$

2.5.2 Interior Loading

Westergaard (1926)

$$\sigma_i = \frac{3\,(1+\mu)P}{2\pi h^2}\left(\ln\frac{l}{b} + 0.6159\right) \tag{2.4 a}$$

$$\Delta_i = \frac{P}{8kl^2}\left\{1 + \frac{1}{2\pi}\left[\ln\left(\frac{a}{2l}\right) - 0.673\right]\left(\frac{a}{l}\right)^2\right\} \tag{2.4 b}$$

2.5.3 Edge Loading

Westergaard (1926)

$$\sigma_e = \frac{0.572P}{h^2}\left[4\log\left(\frac{l}{b}\right) + 0.359\right] \tag{2.5 a}$$

Westergaard's analysis Modified by Teller and Sutherland (1948)

$$\sigma_e = \frac{0.529\,P}{h^2}(1 + 0.54\,\mu)\left[4\log\left(\frac{l}{b}\right) + \log(b) - 0.4048\right] \tag{2.5 b}$$

Ioannides et al (1985) – Semicircular loaded area

$$\sigma_e = \frac{3\,(1+\mu)P}{\pi\,(3+\mu)h^2}\left[\ln\left(\frac{Eh^3}{100ka^4}\right) + 3.84 - \frac{4\mu}{3} + \frac{(1+2\mu)}{2l}\right] \tag{2.5 c}$$

$$\Delta_e = P\left(\sqrt{\frac{2+1.2\mu}{Eh^3k}}\right)\left[1 - \frac{(0.323 + 0.17\mu)a}{l}\right] \tag{2.5 d}$$

When μ = 0.15

$$\sigma_e = \frac{0.803P}{h^2}\left[4\log\left(\frac{l}{a}\right) + 0.282\left(\frac{a}{l}\right) + 0.650\right] \tag{2.5 e}$$

$$\Delta_e = \frac{0.431P}{kl^2}\left[1 - 0.349\left(\frac{a}{l}\right)\right] \tag{2.5 f}$$

Ioannides et al (1985) – Circular loaded area

$$\sigma_e = \frac{3\,(1+\mu)P}{\pi\,(3+\mu)h^2}\left[\ln\left(\frac{Eh^3}{100ka^4}\right) + 1.84 - \frac{4\mu}{3} + \frac{1-\mu}{2} + \frac{1.18(1+2\mu)a}{l}\right] \qquad (2.5\text{ g})$$

$$\Delta_e = P\left(\sqrt{\frac{2+1.2\mu}{Eh^3k}}\right)\left[1 - \frac{(0.76+0.4\mu)a}{l}\right] \qquad (2.5\text{ h})$$

When μ = 0.15

$$\sigma_e = \frac{0.803P}{h^2}\left[4\log\left(\frac{l}{a}\right) + 0.666\left(\frac{a}{l}\right) - 0.034\right] \qquad (2.5\text{ i})$$

$$\Delta_e = \frac{0.431P}{kl^2}\left[1 - 0.82\left(\frac{a}{l}\right)\right] \qquad (2.5\text{ j})$$

σ_c, σ_i, σ_e	Maximum stress at corner, interior and edge loading respectively
Δ_c, Δ_i, Δ_e	Maximum deflection at corner, interior and edge loading respectively
h	Slab thickness
P	Wheel load
K	Modulus of subgrade reaction
a	Radius of wheel load distribution
l	Radius of relative stiffness
b	Radius of resisting section
c	Side length of square contact area = 1.772a
E	Modulus of elasticity of cement concrete
μ	Poisson’s ratio of concrete = 0.15

2.5.4 Dual Tires

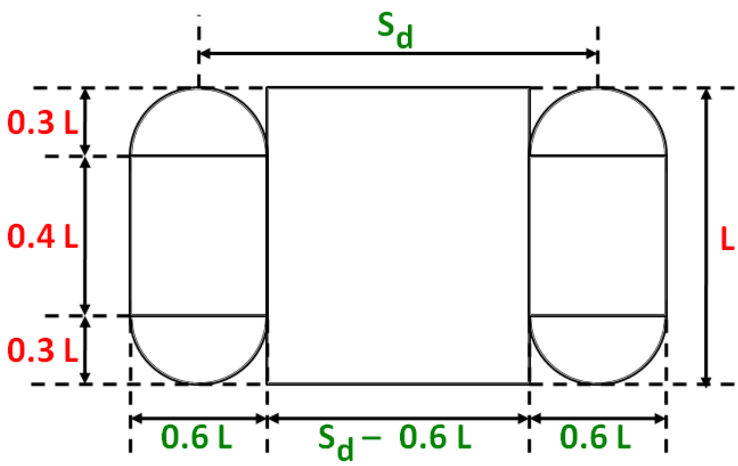


Figure 3.2: Method for Converting Duals into a Circular Area

If P_d is the load on one tire and q is the contact pressure, the area of each tire is

$$\frac{P_d}{q} = [\pi\,(0.3L)^2 + (0.4L)(0.6L)] = 0.5227L^2 \;or\; L = \sqrt{\frac{P_d}{0.5227q}} \qquad (2.6\text{ a})$$

The area of equivalent circle is

$$\pi a^2 = 2(0.5227L^2) + (S_d - 0.6L)L = 0.4454L^2 + S_d L \qquad (2.6\text{ b})$$

The radius of contact area

$$a = \sqrt{\frac{0.8521P_d}{q\pi} + \frac{S_d}{\pi}\left(\frac{P_d}{0.5227q}\right)^{0.5}} \qquad (2.6\text{ c})$$

2.6 Temperature Stresses

2.6.1 <u>Warping Stresses (Westergaard Analysis)</u>

<u>Interior</u>

$$\sigma_{ti} = \frac{E\,\alpha\,t}{2}\left[\frac{C_x + \mu\,C_y}{1-\mu^2}\right] \qquad (2.7\text{ a})$$

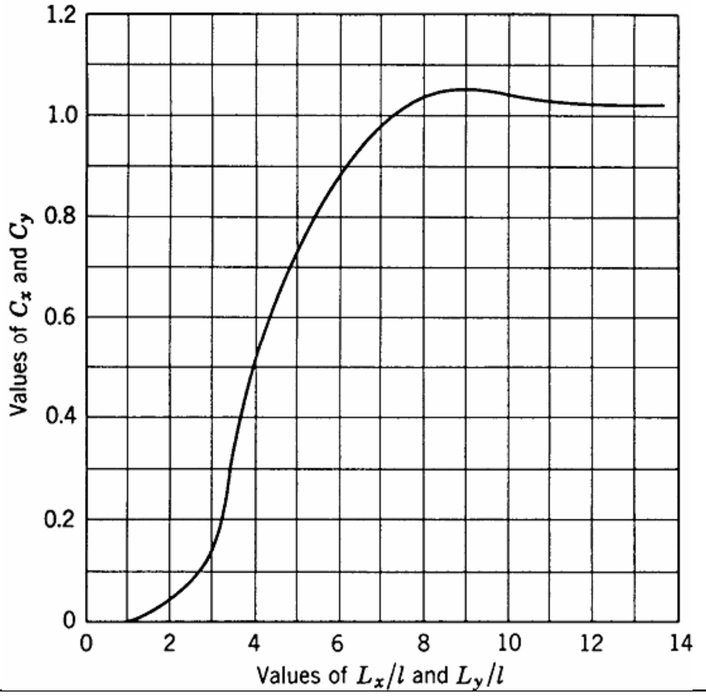
<u>Edge</u>

$$\sigma_{te} = \frac{C_x\,E\,\alpha\,t}{2} \;or\; \sigma_{te} = \frac{C_y\,E\,\alpha\,t}{2} \qquad (2.7\text{ b})$$

<u>Corner</u>

$$\sigma_{tc} = \frac{E\,\alpha\,t}{3(1-\mu)}\sqrt{\frac{a}{l}} \qquad (2.7\text{ c})$$

σ_{tc}, σ_{ti}, σ_{te}	Maximum warping stress at corner, interior and edge region respectively
a	Radius of wheel load distribution
l	Radius of relative stiffness
E	Modulus of elasticity of cement concrete
μ	Poisson's ratio of concrete = 0.15
α	Thermal coefficient of concrete
C_x, C_y,	Bradbury warping stress coefficient



L/l	C	L/l	C
1	0.000	7	1.030
2	0.040	8	1.077
3	0.175	9	1.080
4	0.440	10	1.075
5	0.720	11	1.050
6	0.920	12	1.000

Figure 3.3: Warping Stress Coefficient or Stress Correction Factor for Finite Slab (Bradbury – 1938 and IRC : 58-2002)

2.6.2 Frictional Stresses

$$\sigma_{tf}\, h\, B = B\, \frac{L}{2}\, h\, \gamma_c\, f \qquad (2.8\text{ a})$$

Or

$$\sigma_{tf} = \frac{L}{2}\, \gamma_c\, f \qquad (2.8\text{ b})$$

σ_{tf}	Frictional Stress developed in cement concrete pavement
h	Slab Thickness
B	Slab width
L	Slab length
f	Coefficient of subgrade restraint (maximum value is about 1.5)
γ_c	Unit weight of concrete (about 2400 kg/m^3)

2.7 IRC Recommendations for Design of Plain Jointed Rigid Pavements for Highways (IRC : 58-2002)

2.7.1 Legal Axle Load Limits

Single	10.2 tonnes
Tandem	19.0 tonnes
Tridem	24.0 tonnes

2.7.2 Load Safety Factors

Expressway/NH/SH/MDR	1.2
Lesser importance with lower truck traffic	1.1
Residential and other streets	1.0

2.7.3 Tyre Pressure

Range 0.7 to 1.0 MPa

No significant effect on pavements ≥ 20cm thick

0.8 MPa is adopted

2.7.4 Design Period

Normal – 30 years

Accurate prediction not possible – 20 years

2.7.5 Design Traffic

a. 2-lane 2-way road – 25% of total for fatigue design
b. 4-lane or multi-lane divided traffic – 25% of total traffic in the direction of predominant traffic.
c. New highway links where no traffic data is available - data from roads similar classification and importance
d. Average annual growth rate – 7.5%
e. **Cumulative Number of Repetitions of Axles**

$$C = \frac{365[(1+r)^n - 1]}{r} A \qquad (2.9\ a)$$

$$A = P(1+r)^x \qquad (2.9\ b)$$

A	Initial number of axles per day in the year when the road is operational
R	Annual rate of growth of commercial traffic
N	Design period in years
P	Number of commercial vehicles as per last count
X	Number of years between the last count and the year of completion of construction

2.7.6 Characteristics of Sub-grade

Modulus of sub-grade reaction (K)

a. Pressure sustained per unit deflection
b. Plate bearing test (IS : 9214 – 1974)
c. Limiting design deflection = 1.25mm
d. $K_{75} = 0.5\ k_{30}$
e. One test/km/lane

<u>Approximate K-Value</u>

Approximate K-value corresponding to CBR values for homogeneous soil subgrade

Soaked CBR (%)	2	3	4	5	7	10	15	20	50	100
k-Value (kg/cm^3)	2.1	2.8	3.5	4.2	4.8	5.5	6.2	6.9	14.0	22.2

k-values over Granular and Cemented Sub-bases

k-Value of subgrade (kg/cm^3)	**Effective k (kg/cm^3)**					
	Untreated granular sub-base of thickness in cm			**Cement treated sub-base of thickness in cm**		
	15	**22.5**	**30**	**10**	**15**	**20**
2.8	3.9	4.4	5.3	7.6	10.8	14.1
5.6	6.3	7.5	8.8	12.7	17.3	22.5
8.4	9.2	10.2	11.9	-	-	-

k-value over Dry Lean Concrete Sub-base

k-Value of subgrade (kg/cm^3)	2.1	2.8	4.2	4.8	5.5	6.2
Effective k over 100 mm DLC (kg/cm^3)	5.6	9.7	16.6	20.8	27.8	38.9
Effective k over 150 mm DLC (kg/cm3)	9.7	13.8	20.8	27.7	41.7	-

2.7.7 <u>Characteristics of Concrete</u>

- Modulus of Elasticity
  - Experimentally determined value
  - 3.0×10^5 kg/cm^2 for M40 Concrete
- Poisson's ratio

  $\mu = 0.15$
- Flexural strength of Cement Concrete

  $f_{cr} = 45$ kg/cm^2 for M40 Concrete
- Coefficient of thermal expansion

  $\alpha = 10 \times 10^{-6}$ per °C

2.7.8 Fatigue Behaviour of Cement Concrete

$$N = Unlimited \qquad \textbf{for SR < 0.45} \qquad (2.10\text{ a})$$

$$N = \left[\frac{4.2577}{SR - 0.4325}\right]^{3.268} \qquad \textbf{when } 0.45 \leq SR \leq 0.55 \qquad (2.10\text{ b})$$

$$log_{10}N = \left[\frac{0.9718 - SR}{0.0828}\right] \qquad \textbf{for SR > 0.55} \qquad (2.10\text{ c})$$

N Fatigue life
SR Stress ratio

Stress Ratio and Allowable Repetitions in Cement Concrete

Stress Ratio	Allowable Repetitions	Stress Ratio	Allowable Repetitions	Stress Ratio	Allowable Repetitions
0.45	62,790,761	0.59	40,842	0.73	832
0.46	14,335,236	0.60	30,927	0.74	630
0.47	5,202,474	0.61	23,419	0.75	477
0.48	2,402,754	0.62	17,733	0.76	361
0.49	1,286,914	0.63	13,428	0.77	274
0.50	762,043	0.64	10,168	0.78	207
0.51	485,184	0.65	7,700	0.79	157
0.52	326,334	0.66	5,830	0.80	119
0.53	229,127	0.67	4,415	0.81	90
0.54	166,533	0.68	3,343	0.82	68
0.55	124,526	0.69	2,532	0.83	52
0.56	94,065	0.70	1,917	0.84	39
0.57	71,229	0.71	1,452	0.85	30
0.58	53,937	0.72	1,099	---	----

2.7.9 Stress Computations

Edge Stress

- Due to Load – Picket & Ray's chart
- Due to Temperature –Westergaard's equation (Equation 2.7 b)

Corner Stress

- Due to Load –Westergaard's analysis modified by Kelly (Equation 2.3 c)
- Due to temperature – negligible and hence ignored

2.7.10 Temperature Differential

Recommended Temperature Differentials for Concrete

Zone	States	Temperature Differential, °C in slab of thickness			
		15 cm	20 cm	25 cm	30 cm
I	Punjab, U.P., Uttaranchal, Gujarat, Rajasthan, Haryana and North M.P. Excluding hilly regions.	12.5	13.1	14.3	15.8
II	Bihar, Jharkhand, West Bengal, Assam and Eastern Orissa excluding hilly regions and coastal areas	15.6	16.4	16.6	16.8
III	Maharashtra, Karnataka, South M.P., Chattisgarh, Andhra Pradesh, Western Orissa and North Tamil Nadu, excluding hilly regions and coastal areas	17.3	19.0	20.3	21.0
IV	Kerala and South Tamilnadu excluding hilly regions and coastal areas	15.0	16.4	17.6	18.1
V	Coastal areas bounded by hills	14.6	15.8	16.2	17.0
VI	Coastal areas unbounded by hills	15.5	17.0	19.0	19.2

2.7.11 Recommended Design Procedure for Slab Thickness

- Stipulate design values for the various parameters
- Decide types and spacing between joints
- Select a trial design thickness of pavement
- Compute the repetitions of axle loads of different magnitudes during design period
- Calculate cumulative fatigue damage (CFD)
- If CFD is more than 1.0 revise the thickness
- Check for load+temperature stress at edge with modulus of rupture
- Check for corner stress

2.7.12 Design of Joints

Expansion Joint

If δ' is the maximum expansion in a slab of length L_e with a temperature rise from T_1 to T_2, then δ' = L_e α (T_1 to T_2) where α is the coefficient of thermal expansion of concrete.

Expansion joint gap δ = 2 δ'

Maximum expansion joint gap = 25 mm

Maximum Spacing between expansion joints

for rough interface layer

140 m – all slab thicknesses

for smooth interface layer

when pavement is constructed in summer

90 m – upto 200 mm thick slab

120 m – upto 250 mm thick slab

when pavement is constructed in winter

50 m – upto 200 mm thick slab

60 m – upto 250 mm thick slab

Contraction Joint

$$\sigma_{tc}\, h\, B = B\, \frac{L_c}{2}\, h\, \gamma_c\, f \qquad (2.11)$$

σ_{tc}	Allowable tensile stress in concrete
h	Slab thickness
B	Slab width
L_c	Slab length or spacing b/w contraction joints
γ_c	Unit weight of concrete
f	Coefficient of subgrade restraint (max 1.5)

If Reinforcement is provided, replace LHS by $\sigma_{ts} A_s$

Maximum Spacing between contraction joints

for unreinforced slabs

4.5 m – all slab thicknesses

for reinforced slabs

13 m – for 150 mm thick slab

14 m – for 200 mm thick slab

Dowel Bar Design

- Load transfer capacity of a single dowel bar in

  - **Shear** $$P'_s = 0.785\ d^2 F_s \quad (2.12\ a)$$

  - **Bending** $$P'_f = \frac{2\ d^3 F_f}{L_d + 8.8\ \delta} \quad (2.12\ b)$$

  - **Bearing** $$P'_b = \frac{L_d^2 d F_b}{12.5\ (L_d + 1.5\ \delta)} \quad (2.12\ c)$$

P'	Load transfer capacity of a single dowel bar, kg
d	Diameter of dowel bar, cm
L_d	Total length of embedment of dowel bar, cm
δ	Joint width, cm
F_s	Permissible shear stress in dowel bar, kg/cm^2
F_f	Permissible flexural stress in dowel bar, kg/cm^2
F_b	Permissible bearing stress in concrete, kg/cm^2

- Balanced design for equal capacity in bending and bearing gives

$$L_d = 5d\left[\sqrt{\frac{F_f}{F_b}\left(\frac{L_d + 1.5\ \delta}{L_d + 8.8\ \delta}\right)}\right] \quad (2.13)$$

- Minimum dowel length $L = L_d + \delta$
- Load capacity of dowel system = 40% of wheel load
- Required load capacity factor = $\frac{40\%\ of\ wheel\ load}{(P')_{min}}$
- Effective distance upto which there is load transfer = 1.8 *(radius of relative stiffness)*
- Variation of capacity factor linear from 1.0 under the load to 0.0 at effective distance
- Design spacing = The spacing which conforms to required capacity factor

Recommended Dimensions of Dowel Bars for Rigid Pavements (Axle Load of 10.2t)

Slab thickness, cm	Dowel Bar Details		
	Diameter, mm	Length, mm	Spacing, mm
20	25	500	250
25	25	500	300
30	32	500	300
35	32	500	300

Note : Dowel bars shall not be provided for slabs of less than 15 cm thickness

Tie Bar Design

Area of steel per unit length of joint is obtained by equating the total friction to the total tension developed in the tie bars

$$\sigma_{ts}\, A_s = B\;\; h\, \gamma_c\, f \qquad (2.14)$$

Length of embedment required to develop a bond strength equal to working stress of steel

$$\sigma_{ts}\, A_s = \frac{L_t}{2} P\, \sigma_{bc} \quad \text{or} \quad L_t = \frac{d}{2}\frac{\sigma_{ts}}{\sigma_{bc}} \qquad (2.15)$$

σ_{ts}	Allowable tensile stress in steel = 1400 kg/cm^2	
A_s	Area of tie bar	
B	distance b/w the joint and nearest free edge	
h	Slab thickness	
γ_c	Unit weight of concrete	
f	Coefficient of subgrade restraint (max 1.5)	
L_t	Length of tie bar	
P	Perimeter of tie bar	
d	Diameter of tie bar	
σ_{bc}	Allowable bond stress in concrete	= 24.6 kg/cm^2 for deformed tie bars = 17.5 kg/cm^2 for plain tie bars

Details of Tie Bars for Longitudinal Joint of Two-Lane Rigid Pavements

Slab Thickness cm	Tie bar details, cm				
	Diameter mm	Max. spacing, cm		Minimum Length, cm	
		Plain bars	Deformed bars	Plain bars	Deformed bars
15	8	33	53	44	48
	10	52	83	51	56
20	10	39	62	51	56
	12	56	90	58	64
25	12	45	72	58	64
	16	80	128	72	80
30	12	37	60	58	64
	16	66	106	72	80
35	12	32	51	57	64
	16	57	91	72	80

Note: The recommended details are based on the following values of design parameters

σ_{ts}	Allowable tensile stress in steel	=	2000 kg/cm^2 for deformed bars
		=	1250 kg/cm^2 for plain bars
σ_{bc}	Allowable bond stress in concrete	=	24.6 kg/cm^2 for deformed bars
		=	17.5 kg/cm^2 for plain bars

EDGE LOAD STRESSES IN RIGID PAVEMENT (kg/cm^2)											
K kg/cm^3	Slab Thickness (mm)										
	16	18	20	22	24	26	28	30	32	34	36
SINGLE AXLE LOAD											
Single Axle Load – 6 tons											
6.0	22.490	18.824	16.054	13.902	12.191	10.802	9.656	8.698	7.886	7.191	6.590
8.0	21.457	17.961	15.319	13.264	11.631	10.307	9.215	8.302	7.529	6.868	6.297
10.0	20.684	17.319	14.771	12.790	11.215	9.938	8.886	8.006	7.252	6.625	6.075
15.0	19.331	16.203	13.824	11.972	10.497	9.301	8.317	7.494	6.798	6.203	5.689
30.0	17.131	14.410	12.322	10.684	9.373	8.307	7.427	6.692	6.070	5.539	5.081
Single Axle Load – 8 tons											
6.0	28.615	24.000	20.502	17.779	15.610	13.849	12.396	11.179	10.148	9.264	8.500
8.0	27.246	22.862	19.533	16.939	14.872	13.195	11.811	10.653	9.672	8.832	8.106
10.0	26.216	22.012	18.812	16.315	14.325	12.709	11.376	10.261	9.317	8.509	7.810
15.0	24.405	20.527	17.560	15.236	13.379	11.870	10.626	9.584	8.702	7.948	7.297
30.0	21.450	18.122	15.553	13.524	11.892	10.559	9.454	8.529	7.744	7.073	60494
Single Axle Load – 10 tons											
6.0	34.471	28.971	24.785	21.519	18.912	16.794	15.044	13.578	12.335	11.269	10.347
8.0	32.755	27.552	23.583	20.478	17.999	15.983	14.319	12.925	11.743	10.731	9.855
10.0	31.461	26.488	22.684	19.703	17.320	15.381	13.780	12.439	11.302	10.329	9.487
15.0	29.184	24.623	21.117	18.358	16.146	14.342	12.851	11.601	10.541	9.634	8.851
30.0	25.492	21.604	18.594	16.210	14.284	12.706	11.394	10.291	9.354	8.550	7.856
Single Axle Load – 12 tons											
6.0	40.103	33.774	28.939	25.153	22.126	19.662	17.625	15.917	14.467	13.225	12.150
8.0	38.034	32.067	27.496	23.909	21.037	18.697	16.761	15.138	13.762	12.582	11.562
10.0	36.475	30.785	26.415	22.980	20.225	17.978	16.119	14.559	13.237	12.103	11.123
15.0	33.739	28.537	24.527	21.363	18.817	16.736	15.010	13.561	12.330	11.276	10.364
30.0	29.329	24.918	21.493	18.774	16.575	14.767	13.261	11.992	10.911	9.982	9.178
Single Axle Load – 14 tons											
6.0	45.547	38.432	32.979	28.697	25.267	22.469	20.152	18.208	16.558	15.142	13.917
8.0	43.126	36.434	31.293	27.247	23.999	21.347	19.150	17.306	15.740	14.397	13.235
10.0	41.306	34.933	30.028	26.161	23.053	20.511	18.404	16.634	15.131	13.841	12.725
15.0	38.121	32.307	27.817	24.264	21.407	19.061	17.112	15.472	14.078	12.881	11.845
30.0	32.998	28.101	24.281	21.243	18.783	16.757	15.067	13.640	12.423	11.375	10.466
Single Axle Load – 16 tons											
6.0	50.833	42.964	36.921	32.164	28.344	25.223	22.635	20.461	18.614	17.029	15.656
8.0	48.065	40.675	34.988	30.503	26.895	23.944	21.493	19.434	17.684	16.181	14.880
10.0	45.989	38.957	33.538	29.259	25.812	22.988	20.642	18.668	16.990	15.549	14.301
15.0	42.365	35.961	31.009	27.090	23.925	21.328	19.165	17.342	15.790	14.456	13.299
30.0	36.521	31.173	26.981	23.637	20.923	18.688	16.822	15.244	13.896	12.734	11.724
Single Axle Load – 18 tons											
6.0	55.986	47.388	40.775	35.560	31.364	27.930	25.079	22.680	20.641	18.889	17.371
8.0	52.878	44.810	38.595	33.687	29.732	26.491	23.796	21.528	19.598	17.939	16.502
10.0	50.552	42.879	36.962	32.284	28.511	25.414	22.838	20.668	18.820	17.231	15.853
15.0	46.488	39.520	34.120	29.841	26.383	23.542	21.173	19.174	17.470	16.003	14.729
30.0	39.915	34.147	29.604	25.996	23.009	20.570	18.532	16.808	15.334	14.062	12.956

EDGE LOAD STRESSES IN RIGID PAVEMENT (kg/cm^2)											
K	Slab Thickness (mm)										
kg/cm3	16	18	20	22	24	26	28	30	32	34	36
SINGLE AXLE LOAD											
Single Axle Load – 20 tons											
6.0	61.027	51.719	44.552	38.894	34.333	30.595	27.486	24.869	22.642	20.726	19.066
8.0	57.585	48.856	42.126	36.807	32.516	28.994	26.062	23.591	21.485	19.674	18.104
10.0	55.008	46.716	40.312	35.246	31.155	27.795	24.996	22.634	20.621	18.888	17.385
15.0	50.503	42.996	37.162	32.532	28.789	25.710	23.142	20.972	19.120	17.524	16.137
30.0	43.199	37.031	32.157	28.241	25.048	22.411	20.206	18.339	16.742	15.364	14.164
Single Axle Load – 22 tons											
6.0	65.973	55.968	48.260	42.168	37.254	33.220	29.862	27.030	24.618	22.543	20.743
8.0	62.198	52.825	45.592	39.871	35.251	31.456	28.293	25.623	23.348	21.388	19.686
10.0	59.370	50.478	43.599	38.152	33.752	30.135	27.119	24.571	22.396	20.524	18.897
15.0	54.418	46.397	40.143	35.172	31.149	27.839	25.075	22.739	20.743	19.021	17.524
30.0	46.389	39.836	34.646	30.464	27.045	24.216	21.847	19.841	18.124	16.642	15.350
Single Axle Load – 24 tons											
6.0	70.833	60.147	51.908	45.392	40.131	35.809	32.206	29.165	26.573	24.341	22.402
8.0	66.726	56.727	48.999	42.884	37.943	33.881	30.492	27.630	25.187	23.082	21.252
10.0	63.645	54.174	46.830	41.011	36.307	32.438	29.209	26.480	24.149	22.139	20.391
15.0	58.243	49.729	43.071	37.768	33.470	29.932	26.978	24.479	22.342	20.498	18.892
30.0	49.497	42.573	37.077	32.640	29.004	25.990	23.461	21.318	19.484	17.898	16.517
TANDEM AXLE LOAD											
Tandem Axle Load – 12 tons											
6.0	18.268	15.392	13.278	11.666	10.398	9.368	8.523	7.810	7.201	6.674	6.215
8.0	17.422	14.600	12.535	10.970	9.746	8.763	7.953	7.282	6.706	6.215	5.783
10.0	16.839	14.056	12.023	10.486	9.290	8.336	7.554	6.902	6.355	5.881	5.473
15.0	15.915	13.204	11.222	9.728	8.571	7.653	6.907	6.293	5.777	5.336	4.958
30.0	14.597	12.040	10.154	8.724	7.617	6.742	6.038	5.461	4.981	4.578	4.233
Tandem Axle Load – 16 tons											
6.0	22.993	19.429	16.805	14.801	13.223	11.942	10.888	9.998	9.238	8.577	8.002
8.0	21.873	18.385	15.827	13.883	12.362	11.139	10.133	9.295	8.576	7.964	7.422
10.0	21.096	17.667	15.154	13.248	11.762	10.574	9.603	8.792	8.109	7.518	7.009
15.0	19.854	16.533	14.094	12.248	10.814	9.675	8.750	7.986	7.344	6.795	6.324
30.0	18.075	14.965	12.663	10.914	9.553	8.474	7.603	6.889	6.295	5.793	5.365
Tandem Axle Load – 20 tons											
6.0	27.452	23.265	20.171	17.802	15.932	14.416	13.162	12.105	11.200	10.413	9.727
8.0	26.046	21.963	18.957	16.664	14.864	13.417	12.226	11.230	10.378	9.648	9.004
10.0	25.067	21.064	18.118	15.876	14.122	12.717	11.567	10.606	9.795	9.093	8.488
15.0	23.499	19.636	16.790	14.628	12.943	11.602	10.511	9.605	8.845	8.196	7.636
30.0	21.275	17.661	14.985	12.951	11.365	10.104	9.083	8.244	7.545	6.952	6.447

EDGE LOAD STRESSES IN RIGID PAVEMENT (kg/cm^2)											
K	Slab Thickness (mm)										
kg/cm^3	16	18	20	22	24	26	28	30	32	34	36
TANDEM AXLE LOAD											
Tandem Axle Load – 24 tons											
6.0	31.690	26.936	23.409	20.698	18.554	16.814	15.371	14.153	13.108	12.201	11.408
8.0	29.991	25.369	21.953	19.339	17.280	15.622	14.255	13.108	12.127	11.284	10.543
10.0	28.810	24.284	20.943	18.394	16.394	14.785	13.467	12.365	11.431	10.685	9.926
15.0	26.924	22.558	19.341	16.893	14.979	13.451	12.206	11.170	10.298	9.553	8.909
30.0	24.271	20.190	17.166	14.868	13.076	11.649	10.492	9.539	8.743	8.067	7.490
Tandem Axle Load – 28 tons											
6.0	35.744	30.465	26.537	23.508	21.105	19.153	17.528	16.155	14.977	13.952	13.054
8.0	33.752	28.630	24.834	21.922	19.623	17.765	16.232	14.940	13.838	12.885	12.050
10.0	32.372	27.357	23.651	20.818	18.589	16.791	15.315	14.079	13.027	12.121	11.331
15.0	30.179	25.339	21.773	19.060	16.935	15.235	13.815	12.687	11.711	10.875	10.150
30.0	27.100	22.588	19.239	16.691	14.705	13.124	11.841	10.782	9.897	9.144	8.499
Tandem Axle Load – 32 tons											
6.0	39.642	33.871	29.569	26.242	23.595	21.439	19.641	18.119	16.811	15.672	14.673
8.0	37.364	31.768	27.616	24.427	21.902	19.856	18.163	16.734	15.515	14.457	13.530
10.0	35.790	30.309	26.258	23.159	20.717	18.743	17.117	15.754	14.589	13.587	12.710
15.0	33.296	28.006	24.109	21.144	18.822	16.960	15.438	14.164	13.089	12.167	11.365
30.0	29.783	24.877	21.224	18.438	16.288	14.541	13.139	11.981	11.012	10.188	9.480
Tandem Axle Load – 36 tons											
6.0	43.411	37.172	32.515	28.908	26.030	23.680	21.717	20.051	18.617	17.368	16.268
8.0	40.852	34.801	30.312	26.860	24.123	21.899	20.056	18.495	17.163	16.003	14.987
10.0	39.089	33.161	28.781	25.429	22.785	20.645	18.878	17.394	16.123	15.028	14.067
15.0	36.294	30.579	26.365	23.160	20.649	18.634	16.983	15.604	14.435	13.431	12.557
30.0	32.339	27.069	23.132	20.123	17.777	15.909	14.394	13.143	12.093	11.202	10.434
Tandem Axle Load – 40 tons											
6.0	47.070	40.381	35.385	31.513	28.415	25.881	23.757	21.953	20.398	19.041	17.843
8.0	44.237	37.747	32.934	29.231	26.292	23.899	21.912	20.226	18.785	17.526	16.425
10.0	42.285	35.929	31.232	27.638	24.802	22.504	20.602	19.002	17.629	16.445	15.403
15.0	39.185	33.071	28.555	25.117	22.425	20.264	18.492	17.011	15.753	14.671	13.727
30.0	34.785	29.172	24.972	21.754	19.240	17.237	15.613	14.271	13.145	12.189	11.365
Tandem Axle Load – 42 tons											
6.0	48.864	41.955	36.795	32.793	29.590	26.966	24.766	22.894	21.279	19.870	18.624
8.0	45.894	39.191	34.220	30.396	27.359	24.884	22.828	21.081	19.587	18.280	17.137
10.0	43.848	37.285	32.434	28.721	25.792	23.418	21.451	19.795	18.373	17.146	16.065
15.0	40.593	34.290	29.626	26.076	23.297	21.065	19.233	17.703	16.402	15.282	14.305
30.0	35.972	30.194	25.868	22.550	19.956	17.888	16.210	14.825	13.662	12.675	11.823
Tandem Axle Load – 44 tons											
6.0	50.636	43.511	38.189	34.061	30.754	28.041	25.767	23.829	22.156	20.694	19.401
8.0	47.531	40.618	35.491	31.547	28.415	25.860	23.735	21.929	20.383	19.030	17.845
10.0	45.388	38.624	33.622	29.793	26.772	24.323	22.292	20.581	19.111	17.841	16.721
15.0	41.978	35.490	30.685	27.024	24.157	21.856	19.966	18.388	17.045	15.888	14.878
30.0	37.137	31.197	26.748	23.335	20.662	18.531	16.801	15.371	14.172	13.154	12.275

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