Design Procedures For Dynamically Loaded Foundations

Choice of parameters for equivalent lumped systems

Lumped mass : the mass of the foundation and supported machinery

Damping : ① Geometrical(or radiation) damping – by the decrease in energy density through propagation of elastic waves away from the vicinity of the footing. (Table A-2) (Fig. 7-19)

② internal damping – by energy loss within the soil due to hysteretic and viscous effects. (Table 10-12)

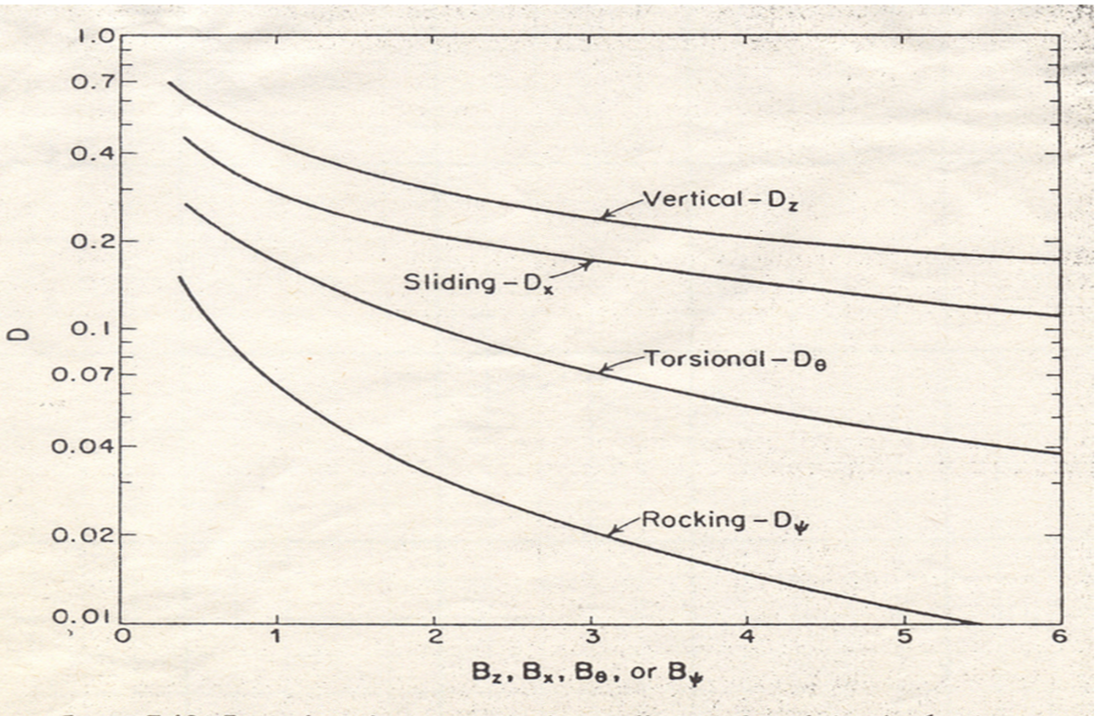


Figure 7-19. Equivalent damping ratio for oscillation of rigid circular footing on the elastic half-space.

- Typical value of the internal damping ratio – 0.05 (Table 10-12)

→ for vertical & sliding mode, negligible (Fig 7-19) (∵geometric damping >>0.05)

→ for torsional & rocking mode, should be included (Fig-7-19)

Table 10-12. Some Typical Values of Internal Damping in Soils

Type Soil	Equivalent D	Reference
Dry sand and gravel	0.03–0.07	Weissmann and Hart (1961)
Dry and saturated sand	0.01–0.03	Hall and Richart (1963)
Dry sand	0.03	Whitman (1963)
Dry and saturated sands and gravels	0.05–0.06	Barkan (1962)
Clay	0.02–0.05	Barkan (1962)
Silty sand	0.03–0.10	Stevens (1966)
Dry sand	0.01–0.03	Hardin (1965)

- Influence of partial embedment – reduction in amplitude on the order of 10~25% depending on the mode of vibration

  → neglection of embedment effect errs on the conservative side

- Influence of the underlying rigid layer increase the amplitude of vibration at resonance

Table A-2. Equivalent Damping Ratio for Rigid Circular Footings

Mode of Vibration	Mass (or Inertia) Ratio	Damping Ratio D
Vertical	$B_z = \frac{(1-\nu)}{4}\frac{m}{\rho r_o^3}$	$D_z = \frac{0.425}{\sqrt{B_z}}$
Sliding	$B_x = \frac{(7-8\nu)}{32(1-\nu)}\frac{m}{\rho r_o^3}$	$D_x = \frac{0.288}{\sqrt{B_x}}$
Rocking	$B_\psi = \frac{3(1-\nu)}{8}\frac{I_\psi}{\rho r_o^5}$	$D_\psi = \frac{0.15}{(1+B_\psi)\sqrt{B_\psi}}$
Torsional	$B_\Theta = \frac{I_\Theta}{\rho r_o^5}$	$D_\Theta = \frac{0.50}{1+2B_\Theta}$

- Spring constant : the most critical factor

  Governs ① The static displacement
  ② magnification factor, M
  ③ the resonant frequency

  Obtained by ① Tests on prototype foundation
  ② Tests on model footings
  (the extrapolation procedure governs the value)
  ③ formulas (Tables 10-13, 10-14)
  (applies to rigid block or mat foundations w/ shallow embedment)

Table 10-13. Spring Constants for Rigid Circular Footing Resting on Elastic Half-Space

Motion	Spring Constant	Reference
Vertical	$k_z = \dfrac{4Gr_o}{1-\nu}$	Timoshenko and Goodier (1951)
Horizontal	$k_x = \dfrac{32(1-\nu)Gr_o}{7-8\nu}$	Bycroft (1956)
Rocking	$k_\psi = \dfrac{8Gr_o^3}{3(1-\nu)}$	Borowicka (1943)
Torsion	$k_\Theta = \frac{16}{3}Gr_o^3$	Reissner and Sagoci (1944)

$\left(Note:\ G = \dfrac{E}{2(1+\nu)}\right)$

Table 10-14. Spring Constants for Rigid Rectangular Footing Resting on Elastic Half-Space

Motion	Spring Constant	Reference
Vertical	$k_z = \dfrac{G}{1-\nu}\beta_z\sqrt{4cd}$	Barkan (1962)
Horizontal	$k_x = 4(1+\nu)G\beta_x\sqrt{cd}$	Barkan (1962)
Rocking	$k_\psi = \dfrac{G}{1-\nu}\beta_\psi 8cd^2$	Gorbunov-Possadov (1961)

(*Note:* values for β_z, β_x, and β_ψ are given in Fig. 10-16 for various values of d/c)

- Elastic constants : G & v

v – cohesionless soils (0.25~0.35) → $\frac{1}{3}$

cohesive soils (0.35~0.45) → 0.40

G – ① From static plate-bearing tests → get k → backcalculate G using formula

② resonant – column test in the lab

③ from the void ratio of the soil & the probable confining pressure $\bar{\sigma}_0$

- For round-grained sands (e < 0.80)

$$G = \frac{2630(2.17 - e)^2}{1 + e}(\bar{\sigma}_0)^{0.5}[lb/in^2]$$

- For angular-grained material (e > 0.6)

$$G = \frac{1230(2.97 - e)^2}{1 + e}(\bar{\sigma}_0)^{0.5}[lb/in^2]$$

(also good for NC clay w/ low surface activity)

④ From the shear wave velocity

$$G = \rho v_s^2$$

- Brief review of other methods or results

DEGEBO(Deutschen Forschungsgesellschaft für Bodenmechanik) :

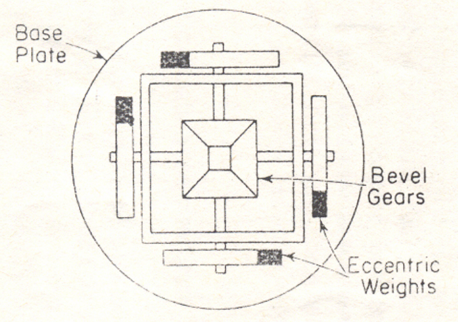


Figure 10-11. DEGEBO Oscillator (top view).

Using a rotating-mass mechanical oscillator (fig 10-11), run extensive number of tests.

- In 1933, reported the followings,

- dynamic response → non-linear
- progressive settlement developed
- dynamic response depends on

① the total weight of the oscillator and base plate
② the area of the base plate
③ dynamic force applied
④ the characteristics of the soil

- established a table for the 'characteristic frequency' for a variety of soils
→ 'natural frequency' of soil (incorrect concept)

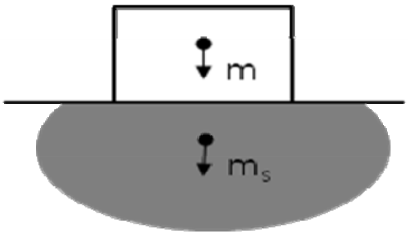
- In 1934, reported on the effect of oscillator weight, base-plate area, and exciting force

- increasing the total weight → lowered the resonant frequency
- increase in the base-plate area → raised the resonant frequency.
- increase in exciting force → lowered resonant frequency.

(this indicates that the soil response is non-linear)

- In-phase mass : a mass of soil moved with the footing
  the resonant frequency

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k}{m+m_s}}$$



m_s depends on

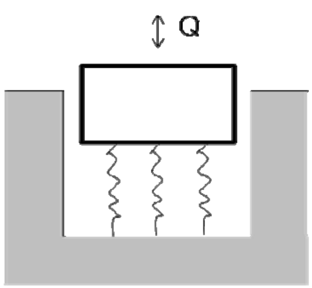
① the dead load
② exciting force
③ base-plate area
④ mode of vibration
⑤ type of soil

- At present, difficult to obtain reliable magnitude of m_s and do not contribute to the evaluation of the amplitude
  → not a significant factor at this stage of development.

- Dynamic subgrade reaction : dynamic subgrade reaction modulus (k') obtained from static repeated loading tests on model footing.

$$k = k'A$$

k : spring constant



- extrapolating formula(Terzaghi, 1955)

  cohesive : $k'_z = k'_{z1}\frac{1}{2d}$

  cohesionless : $k'_z = k'_{z1}(\frac{2d+1}{4d})^2$

  2d=width(or least dimension) of beam, k'_{z1}=least dimension = 1ft

- or Table 10-10

Table 10-10. Recommended Design Values for Subgrade Coefficient k'_z*

Soil Group	Allowable Static Bearing Stress (ton/ft²)	Coefficient k'_z (ton/ft³)
Weak soils (clay and silty clays with sand, in a plastic state; clayey and silty sands)	1.5	95
Soils of medium strength (clays and silty clays with sand, close to the plastic limit; sand)	1.5–3.5	95–155
Strong soils (clay and silty clays with sand, of hard consistency; gravels and gravelly sands, loess and loessial soils)	3.5–5	155–310
Rocks	5	310

* After Barkan (1962).

- Other modes (k'_z = vertical mode)

  Horizontal $k'_x \approx 0.5k'_z$

  Rocking $k'_\psi \approx 2k'_z$

  Torsional $k'_\theta \approx 1.5k'_z$

- Gives no useful information on the amplitude of motion at frequencies near resonance

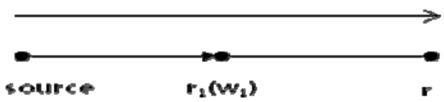
Isolation of foundations

- Mechanical isolation
- Isolation by location
- Isolation by barriers

● Mechanical isolation

use isolation absorbers : rubber, springs, spring-damper system, pneumatic spring

● Isolation by location :



- geometrical damping

$$w = w_1 \sqrt{\frac{r_1}{r}}$$

(where, w : amplitude of motion, r : distance)

(note that $w\sqrt{r} = w_1\sqrt{r_1}$ = constant, ie. no energy loss)

- material damping [∵soil is not perfectly elastic]

$$w = w_1 \sqrt{\frac{r_1}{r}} \exp[-\alpha(r - r_1)], \quad (r > r_1)$$

α : the coefficient of attenuation. (0.01~0.04 (1/ft))

(energy loss due to material damping)

- Isolation by barriers [at least reduction of amplitude to 0.25]

  - Active isolation : isolation at the source
  - Passive isolation : screening at a distance

Examples from practice

- active isolation (covers the area extending to 10 L_R)

  - with trenches fully surrounding the source

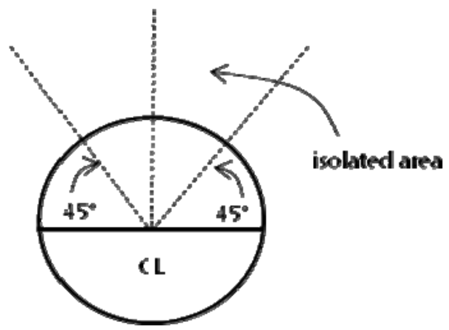
    $H / L_R \geq 0.6$

    (H : trench depth, L_R : Rayleigh wave length($L_R = \frac{v_R}{f}$))

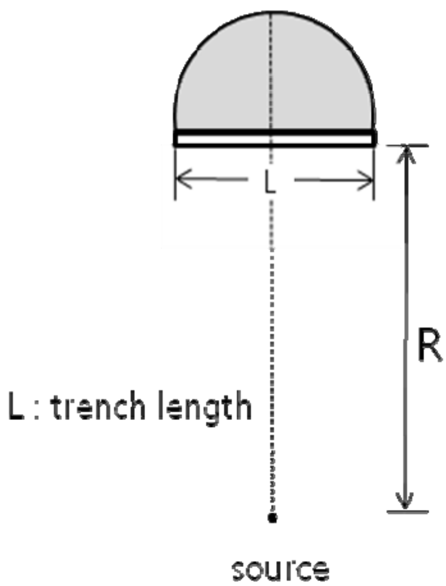
  [note that $H / L_R = 2.0$, not much improvement, i.e. > 0.10]

  - with partially surrounding trenches

    $H / L_R \geq 0.6$



- passive isolation



- $H / L_R \geq 1.33$ (for $2L_R \leq R \leq 7L_R$)
- vertical trench area($H / L_R \times L / L_R$) should be increased as the R increases

ex. For the same degree of isolation
trench area 2.5 at R=2 $L_R \rightarrow$ 6 at R=7 L_R

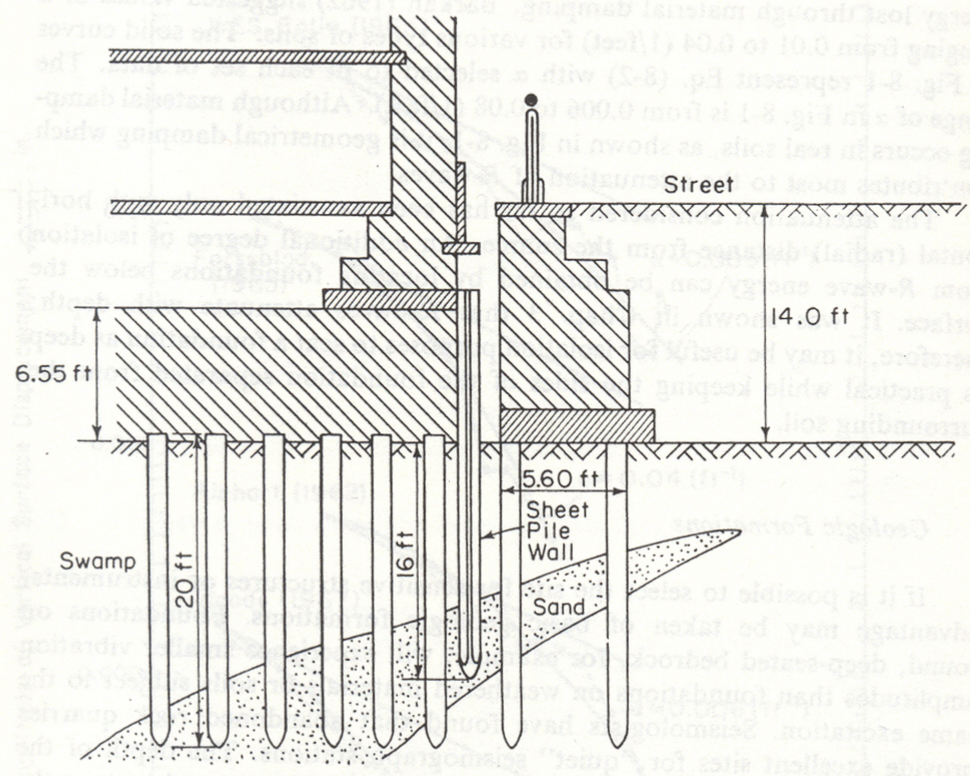


Figure 8-2. Isolation of building from traffic induced vibrations (*after Barkan, 1962*).

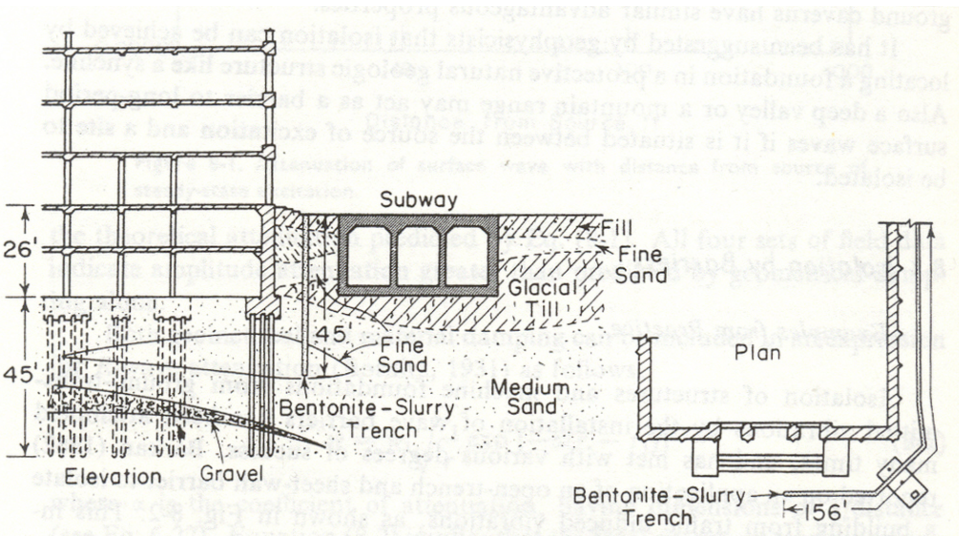


Figure 8-3. Building isolation using bentonite-slurry filled trench (*after Neumeuer, 1963*).

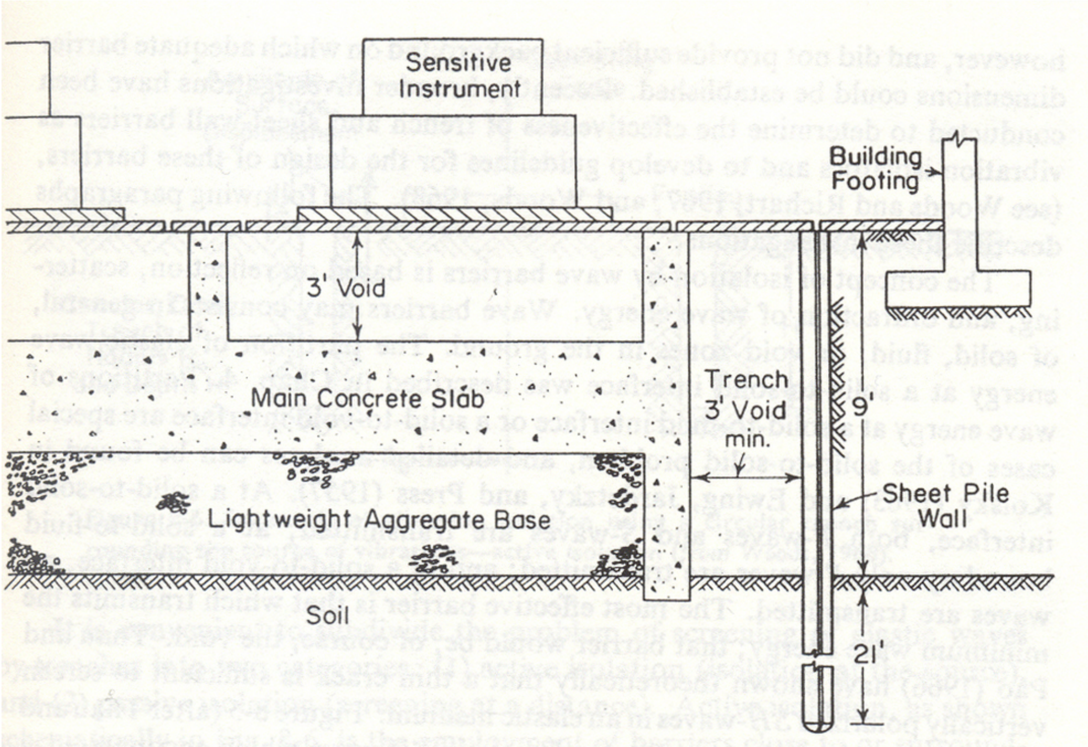


Figure 8-4. Isolation of standards laboratory*(after McNeill et al., 1965)*

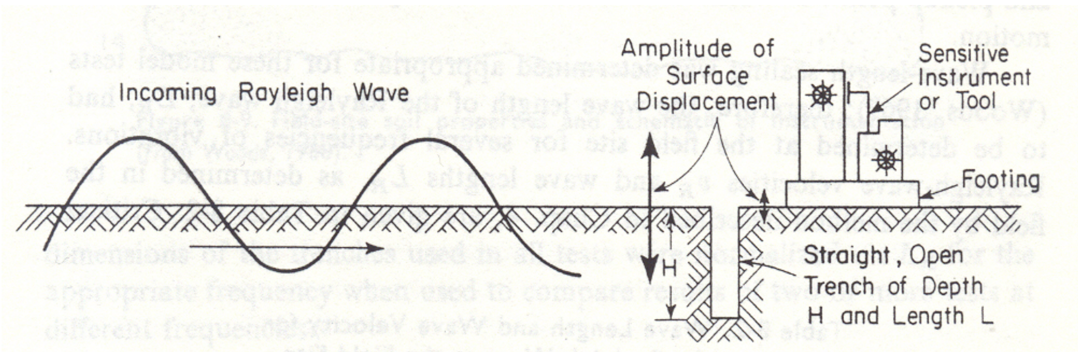


Figure 8-7. Schematic of vibration isolation using a straight trench to create a quiescent zone – passive isolation*(from Woods, 1968)*