01 Jun 1988, 1:00 pm - 5:30 pm

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Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Paper No. 4.28

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SYNOPSIS: During the operation of Hammers and other shock producing machines, strong dynamic effects are generated which depend on the interaction between the different elements of the system. A simple two-degree of freedom system comprising of mass and spring, may offer reasonable result. Better result may be obtained by Wave Equation approach. This paper compares these two numerical schemes with the observed behavior of one Belt-drop stamping hammer.

INTRODUCTION

Satisfactory performance of Hammer foundation plays a vital role in any industrial complex. Lack of proper attention in the design consideration may lead to excessive vibration, settlement, cracking and may create trouble to the adjacent machine or structure. The disaster may sometime force a shutdown of production, incurring tremendous financial losses. The designer should be well aware of the limitation of his idealized model and the deviation from the reality should be taken into account by his engineering judgment.

Hammers are shock producing machines which generates strong shock pulses of short duration. A typical Hammer foundation consists of (i)frame, (ii)head, (iii)anvil and pad, (iv) foundation block, embedded in soil. Depending upon individual case and local soil condition designers may make some modification of this basic assemblage. The powerful blows generated by the fall (either free or energized with steam) of the hammer travels through foundation and soil. Only a small part of this impact energy is utilized for the plastic deformation of the material being forged. The general objective of the design is to achieve vibration amplitude, settlement and stresses within acceptable limit and the velocity, acceleration of the propagated noise should be much below the human perceptibility level in the near vicinity.

System identification

Designer many times faces difficulty in identifying the system to his satisfaction. The shape and duration of the pulse plays an important role in the analysis. In the routine design procedure the real pulse is idealized as a short duration rectangular pulse for which Duhamel integral offers solution for a single degree of freedom system. With common Hammer the pulse duration is in the range of .01 to .02 sec and for special type of hammer with severe shock, it may come down to .001 to .002 sec. It seems that rectangular pulse idealization is justified.The simplest mathematical model is a mass resting on a spring and dashpot. Role of soil is taken into account by the stiffness of the spring and damping of the dashpot. For a symmetric foundation with centric blow, a single degree of freedom system may well represent the vertical vibration of the system but for eccentric blows on asymmetric foundation the number of degrees of freedom may be higher for a near real representation of the assembly. Barkan's [1] method of analysis based on Winkler's model which replaces the soil by a series of elastic and independent springs on rigid base. This model has been successfully applied in static cases. But in the dynamic case, energy dissipation through radiation damping can not be accommodated in a Winkler type model. It is hoped that for a very low frequency periodic loading, near static, this model may offer reasonable estimation. Deletion of damping in this model overestimates the amplitude of vibration. The Winkler-Voight model, where soil is idealized as a series of independent elastic springs with a series of independent linear dashpots in parallel, tries to consider damping but as a velocity dependent quantity only. The results obtaind from model tests, when backfigured to accommodate material and geometric damping through the coefficient of viscosity of velocity dependent dashpot, the stiffness coefficient of the spring shows discrepancy with other test like repeated loading test. This confusion may lead a designer to think that,'the model conspicuously lacks what all models should possess -- predictive power' [2]. Lysmer analogue of frequency dependent spring-dashpot assembly offers confidence by its reasonable predictive capability. Since then many more methods have come up based on the engineering application of half-space theory, frequency domain analysis of simplified models based on impedance or compliance function method [4] While frequency domain approach is a strong tool for economic analysis of vibration problem, the time-domain approach allows one to consider the effect of hysterisis of memory type soil element. Many complicated and typical behaviors could be considered reasonably through discrete idealization of the governing differential equation and soil behavior. Especially the effect of hysterisis loss in loading-unloading cycle and no-tension, a very significant property of soil, could be well studied in the time-domain analysis.

Summing up, the different analytical procedure, within the ambit of success or failure, is best left to individual perception. Far too many factors were at play to defy a definitive description. But the final judgement comes from the real performance of Hammer foundation. In this study, the actual field observed performance of a Belt drop Stamping Hammer(Fig.1) was compared with (i) the predicted value of a simplified single degree of freedom mass-spring system and (ii) the numerically predicted value based on time-domain shock propagation analysis with elasto-plastic, no-tension soil.

Fig. 1 Hammer Assembly

Geotechnical condition

A detailed soil investigation was conducted at site. Two bore holes were advancd upto 9m depth and Standard penetration tests was conduced at 1.5m interval as per IS: 2131-1981. Representative samples collected from the sampler were later used for the determination of natural moisture content and classification. It was found that the stratification is uniform in nature. A typical bore log with SPT values is presented in Table 1.The soil is cohesionless, classified as silty-sand and poorly graded sand (SM-SP) upto the depth of investigation. The percentage of fines decreas with depth. The position of steady water table was at 6m below ground. Average field densities of the soil above and below ground water table were found as 1.49 t/m^3 respectively. The SPT value of top 3m soil is 10 but increses to 20 between 3 to 7m. Below this depth there exists a still stiffer layer. The safe static bearing capacity of soil, at a depth of 3m below ground, for a base size 2.5mx5.6m, obtained from N-values, was $23 t/m^2$ from, 25mm allowable settlement criterion. After application of water table correction the allowable soil pressure comes out as $11.5 t/m^2$.

Forced vertical and horizontal vibration tests, were conducted on two 1.5mx.75m concrete blocks, at a depth 3m below ground, to evaluate dynamic coefficient of elastic uniform compression and uniform shear under periodic loading. The tests were conducted by exciting the block using a mechanical oscillator and a speed control D.C. motor to drive it. The oscillator motor assembly was rigidly fixed on top of the block and sinusoidal vertical unbalanced force of varrying magnitudes were generated by controlling the eccentricity of masses in steps. The vibration of the block were picked with a Miller type accelerometer, mounted on the top face of the block, to sense the vertical vibration, amplified through a universal amplifier and recorded directly on ink-writing oscillograph. The records were obtained for different frequencies of excitations by controlling the speed of the driving motor. For forced horizontal test the oscillator was mounted to generate a sinusoidally varying force in horizontal direction on the top of the block and recording was done with 3 accelerometers mounted on vertical face of the block to record horizontal motion under dynamic loading. It was observed that the resonant frequency was around 900rpm in vertical and around 600rpm in horizontal direction. Fig 3 shows a typical amplitude-frequency plot.

Fig.2 Oscillator mounting

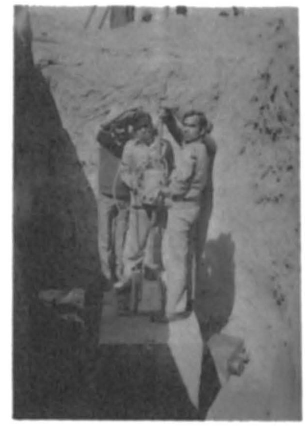

Fig. 4 Impact test

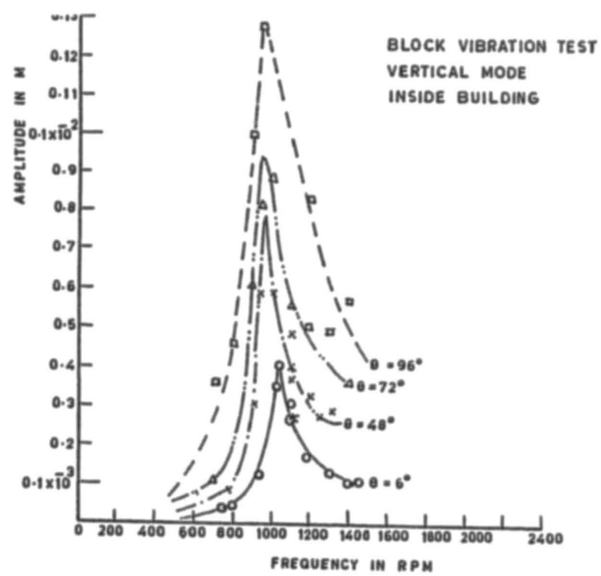

Fig. 3 Frequency-Amplitude Plot

A vertical Impact test, on the same block, was conducted to evaluate the multiplying factor for the dynamic properties of soil under impact. A wooden block of .40mx.30mx.20m thick was centrally mounted firmly by nuts and bolts on the concrete block (Fig.4). The 65 kg. hammer was dropped freely on the wooden block from a height of 0.5m, through a vertically held guide rod on the block. The rebound of the hammer was seen to be 2cm. The transient motion of the blcok, undder this vertical impact, was noted in a pen recorder through accelerometer mounted on the block. This record was analyzed for evaluating the multiplying factor of the dynamic soil properties under impact.

From the Impact test, Forced vibration test and Bore long stratification data it was concluded that the following dynamic soil properties may be adopted for the analysis:

i) Coefficient of elastic uniform compression = $1.6 kg/cm^3$
ii) Coefficient of elastic uniform shear = $0.8 kg/cm^3$
iii) Multiplying factor = 2.5

Fig. 5 Instrumented Hammer Fouldation

Performance study of the Hammer Block

The design data of the hammer assembly is presented in Table 2. The foundation of the hammer was 5.6mx4.5mx2.6m deep with one layer of brick wall all round, fully embedded in soil. Accelerometer pick-ups were mounted on the foundation block in horizontal and vertical directions. Wooden pegs were placed on ground upto a distance of 15m from the wall of the foundation. Acceleration pick-ups were mounted on each peg, in turn, to record the geometric attenuation. Simultaneous recording of 4 pick-ups, 2 of foundation block and 2 on pegs on ground, were taken for comparison. The reduction ratio (Acceleration on ground to Acceleration on block) shows almost an exponential decay with distance. Amplification takes place only within a close distance of 3.5m. The wavelet which travels out along surface shows Richer type wave shape which is typical of Visco-Elastic medium.

Analysis based on simple model

A simplified two degree mathematical model was considered for the preliminary analysis.

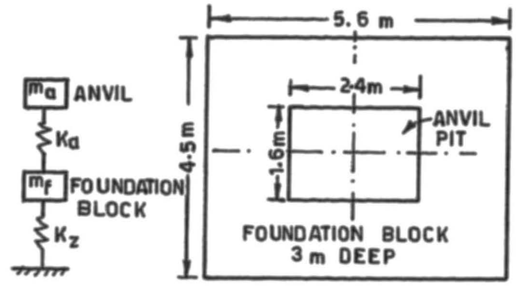

Fig. 6 Mathematical model and plan area

Table 2. Design Data

1. Weight of tup (Wt) = 1,000kg
2. Weight of anvil (Wa) = 18,000kg
3. Weight of frame (Wf) = 24,221kg
4. Height of fall of tup (h) = 1.6m
5. Area of avil base (2.2mx1.1m) (Aa) = $2.4m^2$
6. Weight of foundation block (5.6mx4.5mx3mx2.4) (Wb) = 181.44t
7. Mass of foundation block (mb) = $18.1 t\text{-}sec^2/m$
8. Coefficient of restitution (k) = 0.25
9. Bearing capacity of soil (σ)= $11.5 t/m^2$.
10. Coefficient of elastic compression of soil(2.5x1.6)(Cs) = $4 kg/cm^3$
11. Thickness of rubber pad under anvil (ta) = .04m
12. Area of the rubber pad (2.2mx1.1m) = $2.42m^2$
13. Area ratio, 2.42/(2x4x(2.2+1.1))(Ar) = 9.17
14. Stiffness of the rubber pad for this area ratio = $16.4\times10^6 kg/cm^3$
15. Shore hardness of the rubber pad = 55
16. Shear modulus of the rubber pad = $8.26 kg/cm^2$
17. Youngs modulus of the rubber pad = $33.1 kg/cm^2$
18. Bulk modulous of the rubber pad = $10{,}000 kg/cm^2$

Essential computations were carried out following Barkan[1]

Mass of the anvil (ma)=Wa/g=18.36 $kg\text{-}sec^2/cm$.
Mass of the frame (mst)=24.7 $kg\text{-}sec^2/cm$
Mass of the foundation block (mf)=185.14 $kg\text{-}sec^2/cm$
Mass of the frame and foundation block (m1)=209.8 $kg\text{-}sec^2/cm$
Stiffness of the rubber pad (ka)=16×10^6 kg/cm
Stiffness of the foundation soil=(kz)=$4\times(5.6\times4.5)\times10^4=10^6$ kg/cm
Falling velocity of tup (v)=$\sqrt{2gh}$=560cm/sec
Velocity of anvil after impact (V)=(1+k)v/(1+W/Wt)= $1.25\times560/(1+18\times10^3/10^3)$=37cm/sec
The frequency of the natural vibration of the anvil (wa), assuming the soil to be rigid (kz=∞) is given as $wa^2=ka/ma=87\times10^4\ sec^{-2}$. Thesquare of other limiting frequency (wa) of the entire system, assuming ka= ∞ ,is given by $wz^2=kz/(mf+ma+mst)=4.4\times10^4 sec^{-2}$ The ratio α =ma/(mf+mst)=.087
Frequency equation, $wn^4-(wa^2+wz^2)(1+\alpha)wn^2+(1+\alpha)wa^2.wz^2=0$
when solved for wn, offers $wn1=9.734\times10^2$ sec-1,wn2=66.4sec-1
Amplitude (af) of the foundation is obtained from
$af=-(wa^2-wn2^2)(wa^2-wn1^2)V/[wa^2(wn1^2-wn2^2)wn^2]$=.0483cm
Amplitude (Aa)of the anvil is obtained from

$aa = -(wa^2 - wn1^2)V/[(wn1^2 - wn2^2)wn^2] = .0485cm$

Acceleration of the foundation block is obtained as $\ddot{af}$ (neglecting the contribution from lower natural frequency wn2, $\ddot{af} = .0483 \times 9.734 \times 100 \times 66.4 = 3.12g$.

Observed vertical acceleration of the foundation block was 2.2g.

Numercial modelling of the Hammer Foundation

Discrete idealization of space-time frame [5] has been adopted here. The hammer foundation block has been idealized as an assemblage of lumped connected weights W (2) through W(p) which are connected by weightless springs of stiffness K (1) through K (p-1). Soil surrounding each element offers visco-elasto-plastic resistance in shear and the first anvil element of W (1) and the tip soil element offers resistance in compression only (i.e., no tension). The time domain was also discretised into small interval.

Essentially, the system is considered to be complosed of (1) anvil (2) foundation mass (3)side soil shear layer (4) no-tension tip soil.

The finite difference form of the numerical scheme is as follows:

$$D(m,t) = D(m,t-1)+V(m,t-1).dt$$

$$C(m,t)=D(m,t)-D(m+t,t)$$

$$F(m,t)=C(m,t).KF(m)$$

$$R(m,t)=(D(m,t)-D(m,t)).KS+q.KS.J(m).V(m,t-1)$$

$$V(m,t)=V(m,t-1)+(F(m-1,t)-F(m,t)-R(m,t)).g.dt/W(m)$$

Where, D,C,F,R.V are displacement, compression and force of internal spring, total soil reistance and velocity respectively of m foundation element at time t. Kf and Ks are foundation and soil stiffness modulous. J is the damping coefficient, q is the plastic yielding limit of soil element and dt is the time step. The displacements and velocities are found out at the end of full interval (n). dt, (n+1).dt etc. but forces and acceleration are obtained at half time intervals (n+1/2)dt, (n+3/2)dt etc. For tip soil element when $(D(m,t)-D(m,t)) < 0, KS=0$. The numercial scheme sweeps over time with the known impact velocity of anvil, the first element, and with initial at-rest condition for all other foundation elements. The travel time needed for elastic wave to pass through one foundation element was taken as time step dt in this scheme.

The hammer foundation problem was studied through this numcercial scheme. The block was divided in 10 elements and Novak's frequency independent stiffness parameters [2] were suitably selected for soil spring and damping constants.

Fig. 7 shows Acceleration-time response of the block. Though the numerically computed value of vertical acceleration (2.8g) is close to observed value 2.2g, the shape of this acceleration response curve is different and decay is fast. It has been observed through parametric study the shape of the acceleration pulse depends on the impact pulse input data and it is very difficult to assess the nature of this.

CONCLUSION

The field study and numerical analysis on the vibration of Hammer foundation problem may lead to following conclusions.

1) Prototype impact test may assess well the dynamic properties of soil which are essential parameters for hammer foundation design.

2) Two-degree freedom system consisting of mass and spring may offer reasonable result.

3) Numerical scheme of wave equation approach offers better result provided the time history of the impact force is correctly assessed.

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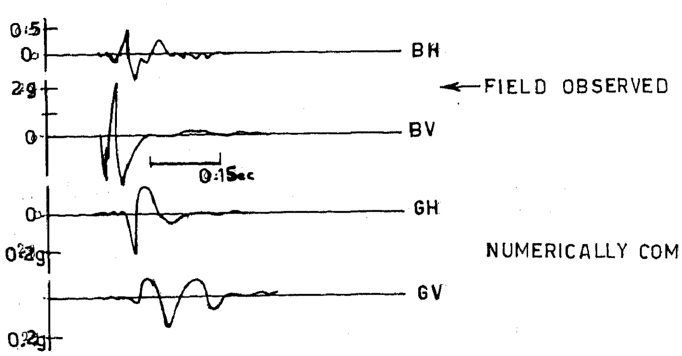

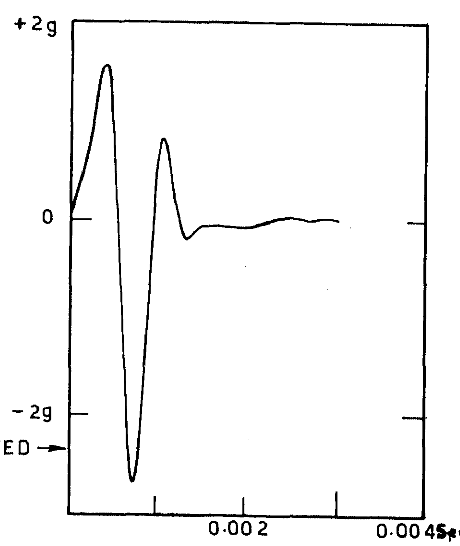

FIG. 7 _ ACCELERATION TIME RESPONSE

TABLE _ 1

DEPTH (m)	I. S. CLASSIFICATION DESCRIPTION	HATCHING	SAND %	FINES %	Wn %	WPL %	WLL %	WATER LEVEL	N VALUE
0									—
1·5	SILTY SAND SM—SP (NON - PLASTIC)		88·05	11·95	5·59	—	—		12
3·0	POORLY GRADED SAND SP		95·00	5·00	3·95	—	—		15
4·5			95·60	4·40	3·51	—	—		21
6·0			94·95	5·05	22·99	—	—	▽	22
7·5	SILTY SAND SP—SM (NON-PLASTIC)		93·95	6·05	23·80	—	—		38
9·0	POORLY GRADED SAND SP		95·50	4·50	22·51	—	—		41

BORE LOG FOR SOIL STRATA AT BH-2

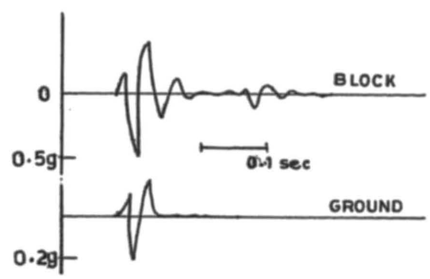

FIG. 8 _ IMPACT TEST RESPONSE

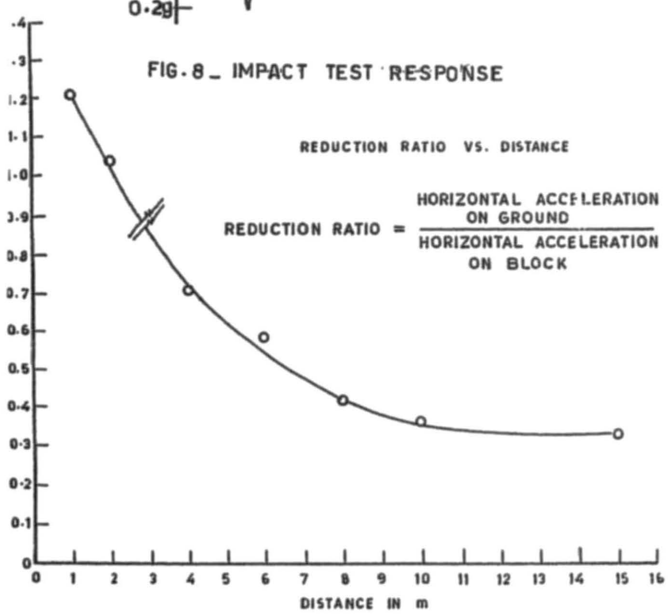

FIG. 9 _ATTENUATION OF ACCELERATION RATIO WITH DISTANCE FROM SOURCE

FIG. 10 _ PICKUP ON FOUNDATION

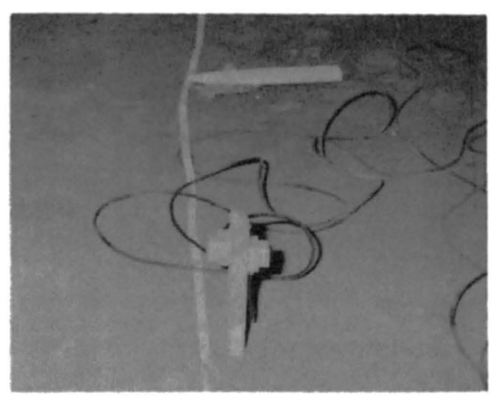

FIG. 11 _ PICKUP ON GROUND