Aseismic design of expanded poly-styrol fill in Japan

Hikaru Hotta: Construction Projects Consultants, Inc.
YSK Bldg., 3-23-1 Takadanobaba, Shinjuku-ku, Tokyo, Japan
E-mail: hotta@cpcinc.co.jp

Abstract
Studies have been conducted urgently in earthquake-prone Japan to establish an aseismic design method for expanded poly-styrol (EPS) fill since the introduction of the EPS construction method. Both laboratory and field verification studies have been carried out mainly by institutions, and numerous papers have been published for establishing aseismic methods. Seismic behavior of EPS fill has gradually been identified. Aseismic design methods for strong ground motions have been established. This paper presents some of the study results and existing aseismic design methods for EPS fill.

Keywords: EPS, aseismic performance of EPS fill, aseismic design methods, shake-table tests, field observation, dynamic characteristics

1.Introduction
Studies have been conducted urgently in earthquake-prone Japan to establish an aseismic design method for expanded poly-styrol (EPS) fill since the introduction of the EPS construction method. Both laboratory and field verification studies have been carried out mainly by institutions, and numerous papers have been published for establishing aseismic methods. Seismic behavior of EPS fill has gradually been identified. Aseismic design methods for strong ground motions have been established.
Double vertical wall type EPS fill has recently been constructed in Japan in a rapidly increasing number of cases and on larger scales in narrow valleys in mountainous areas and behind bridge abutments. Design of EPS fill is often required to ensure safety against inland earthquakes such as the Hyogo-ken-Nanbu earthquake of 1995. During the design under the above conditions, aseismic performance of EPS fill has been highly evaluated. This paper presents the results of studies made so far, under the following subjects.

- Study of seismic behavior of EPS fill based on field observation [1),2)]
- Study of dynamic deformation and strength characteristics of EPS fill material by indoor dynamic element tests [3)~ 8)]
- Result of shake-table tests on EPS fill for road widening [9),10),11)]
- Aseismic design methods for great earthquakes [12)]

2. EPS fill subjected to hysteretic seismic forces[1),2)]
In recent years, great earthquakes have occurred in various parts of Japan, and seismic damage to EPS fill has been identified. Table 1 lists great earthquakes and EPS fill constructed near their focus.
No direct damage to EPS fill has been recognized during the great earthquakes. Aseismic performance of the EPS fill is therefore considered extremely high. Most of the EPS fills subjected to hysteretic seismic forces were not designed to resist earthquakes. No instruments were installed to identify their behavior during earthquakes, either. In the future, systems for monitoring the behavior of EPS fill during a disaster, especially an earthquake, need to be enhanced to supplement regular monitoring in normal times. Outlined below are the results of investigations of the damage to specific EPS fill near the focus during respective earthquakes.
(1) Kushiro-oki earthquake of January 15, 1993 (magnitude: 7.8, seismic intensity: VI in Kushiro)
The earthquake caused slope failures and severe damage to roads, levees and port facilities in Kushiro City. About 8-km north-east to the city center, EPS fill had been constructed behind the abutment of the Senmoh overbridge in Kushiro-cho above national highway route No. 44 to control the settlement of soft ground. A profile of the EPS fill is shown in Figure 1. The EPS fill was constructed for a depth of 2.5 m above the retaining wall of a height of about 6 m. Piles reached the bedrock at a depth of 52 m below the surface. In the fill of this shape, cracking in the pavement at the boundary between existing and EPS fills had been of concern due to an increase in moment acting on the pile foundations of the retaining wall, and the difference in mode of response of the two types of fill. No seismic damage due to strong ground motion was, however, recognized in the EPS fill. The Setsuri Bridge in the vicinity on national highway route No. 44 suffered slippage of abutments and 35-cm settlement of backfill for abutments, which proved the aseismic performance of the EPS fill was satisfactory.

(2) Notohanto-oki earthquake of February 7, 1993 (magnitude: 6.6, seismic intensity: V in Wajima)
EPS fill had been constructed to prevent landslides on national highway route No. 249 in the Semmaida district in Wajima City, Ishikawa prefecture that was closer to the focus than Wajima where seismic intensity of V was registered (a peak horizontal acceleration of 130 gal was measured at the Wajima observatory). A profile of the fill and the construction site relative to the focus are shown in Figure 2.
The earthquake had its focus off the coast of the Noto Peninsula, about 40 km north-east from the site of construction. Liquefaction caused by the earthquake was observed in various points on the peninsula. At the site with landslide-prone topography, damage by ground motions had been of concern. Investigations of seismic damage found slope failures in the surrounding ground but no damage to the EPS fill due to ground motions.[2)]

(3) Hokkaido-nansei-oki earthquake of July 12, 1993 (magnitude: 7.8, seismic intensity: V in Esashi)
EPS fill had been constructed behind a box culvert in the fill for road widening to control settlement of the box culvert in Ajigasawa near Fukaura, Aomori prefecture where a seismic intensity of x was recorded during the earthquake. Cracking was found on the road surface due to long-term settlement but no disturbance was observed in the fill for road widening.

(4) Hokkaidotoho-oki earthquake of October 4, 1994 (magnitude: 8.1, seismic intensity: VI in Kushiro)
Seismic damage was investigated in the EPS fill behind the abutment of the Senmoh overbridge where investigations were also made during the Kushiro-oki earthquake described in (1) above. No direct damage by the earthquake was found.

(5) Hyogo-ken-Nanbu earthquake of January 17, 1995 (magnitude: 7.2, seismic intensity: VII in Hanshin area (Osaka and Kobe)) The earthquake registered a seismic intensity of VII at the sites where EPS fill had been constructed (Figure 3). Much greater ground motions may have occurred than those during the earthquakes mentioned above. Seismic damage was investigated at the ten sites shown in Table 2 where EPS fill may have been subjected to strong ground motions during the earthquake. Such a large number of sites were investigated not because numerous EPS fills had been constructed in the district but because the earthquake induced strong ground motions over a wide area. Seismic damage investigations found that the EPS fills caused no direct damage due to ground motions at any of the sites, but that EPS fill was deformed due to the deformation of surrounding structures at one site due to the reduction of the bearing capacity of the foundation ground as a result of liquefaction in the foundation ground under the structures (No. 10 in Table 2). In the area around the site of deformation, severe seismic damage was confirmed to port structures, bridges and buildings. Deformation was found in the EPS fill at the approach to a bridge, but the bridge remained in service immediately after the earthquake.

3. Existing shake-table test results

3.1 Dynamic strength and deformation characteristics

In soil engineering, the introduction of dynamic response analysis for soil made dynamic deformation characteristics in the zone of very small strain necessary. Studies have therefore been conducted energetically on dynamic deformation characteristics of soil for the past dozen years. This section presents some of the existing studies on dynamic characteristics of EPS fill material.

(1) Dynamic strength characteristics
Tateyama et al.[5),8)] investigated changes in strength and deformation characteristics of EPS blocks during a maximum of four million times of uniaxial cyclic loading. The test focused on the fatigue characteristics of material. Loading was applied cyclically while varying the stress and frequency. The specimen was a cube 50 cm on a side. Figure 4 shows the test results for a unit weight of 20 kgf/m^3. It was verified that dynamic cyclic loading had no effect until the number of cycles reached four million even when the ratio of the dynamic loading stress to unconfined compressive strength (referred to as loading ratio below) was 1.0.[8)]
(2) Dynamic characteristics
Tamura[6)] obtained the Young's moduli, Poisson's ratios and damping coefficients of the materials used in model tests by seismic refraction and free vibration tests. As a result of application of vibration on the surface of an EPS block 182 cm x 90 cm x 40 cm for measuring elastic wave propagation velocity, a velocity of P wave propagating in the block Vp of 714 m/sec and an S wave velocity Vs of 484 m/sec were obtained. Then, a Young's modulus E of 108 kgf/cm^2 and a Poisson's ratio v of 0.075 were obtained accordingly. For free vibration testing, a 80-cm-long cantilever of rectangular cross section was used. A

Young's modulus of 114 to 128 kfg/m^2 was obtained from the wave of deformation at the end of the beam subjected to vibration. It was verified that damping coefficient deceased with a reduction in amplitude. An extremely low damping coefficient of about 0.9% was confirmed (Figure 5).

(3) Dynamic deformation characteristics

An example of a typical dynamic deformation test using an EPS block is described below. [7)]

Compressive tests for EPS generally use cubic specimens as specified in JIS K-7220 standards. Tests were therefore conducted using cubic specimens 50 mm x 50 mm and solid cylindrical specimens that are commonly used in soil tests, to identify the effects on the dynamic deformation characteristics according to varying a. shape (prism or cylinder), b. size and c. diameter-height ratio (D/H) of the specimen. The following characteristics were confirmed (Fig. 6 through 7).

(i) Both shear stiffness G and damping coefficient were not so dependent upon shea r strain. Damping coefficient was independent of confining pressure and ranged from 1 to 3%.

(ii) Shear stiffness G was higher for prism specimens than cylindrical specimens regardless of shear strain range.

(iii) Initial shear stiffness Go decre ased with increases in confining pressure as static strength decreases with increases in confining pressure under a static condition. Thus, a similar trend was confirmed for dynamic deformation.

3.2 Shaking test

Model shaking tests have been frequently conducted to identify the seismic response characteristics and stability of EPS fill. Presented below is a model shaking test on EPS fill for road widening conducted by the Public Works Research Institute of the Ministry of Construction. [10),11)]

(i) Test details

Seven models were made of the EPS fill on a slope (Figure 8), and a horizontal shaking test was conducted in two to four cases under different conditions. The objectives of respective models and details of test cases are listed in Table 3.

(ii) Test method

Shaking conditions are listed in Table 4. In step 1, acceleration response magnification factor of the overburden was measured while varying input frequency to examine the frequency response of the model. In steps 2 and 3, the response acceleration and response displacement of each portion of the model, earth pressure applied by the backfill material to the EPS fill, and anchor tensile force during shaking were measured to examine the response characteristics and failure properties during strong shaking, and the deformation of the model was recorded.

(iii) Test results and discussion

-Effects of metal connectors

For model 1, tests were carried out when metal connectors were used for connecting EPS blocks and when no metal connectors were used. It was confirmed that metal connectors were effective in preventing sliding between blocks and had little effect on frequency response during shaking.

-Effects of soil grade

The frequency response characteristics obtained in step 1 in the case with a small overburden load and no anchors for models 2 to 4 with a varying soil grade are shown in Figure 9. Small effects of soil grade on the resonant frequency and response magnification factor were confirmed.

The mean friction factor オ effective on the bottom o f the EPS fill obtained by back calculation from the horizontal response acceleration of each portion of model 4, and the degree of sliding at the bottom end of the EPS fill are compared in Figure 10. The figure shows that sliding started at a friction factor of about 0.5, which was almost identical to the friction factor between EPS blocks obtained by static tests. The safety factor against overturning obtained by back calculation as a proportion to the angular moment with the center at the toe of the EPS fill was 1.0 or higher for any model when downward vertical inertia acting on the overburden was taken into consideration. This corresponds to the fact that no failure in the overturning mode occurred in tests.

-Anchor tensile force

When the soil has sufficient stiffness, the anchor is expected to carry larger forces.

-Earth pressure of backfill material

Relationships between the maximum value in compression of the resultant of the dynamic lateral earth pressure acting from the backfill material on respective stages on the EPS fill (a variance during shaking), and the response acceleration of the backfill material for models 6 and 7 in cases without anchors are shown in Figure 11. Dynamic earth pressures were almost the same at any stage regardless of the volume of backfill material, and increased with the response acceleration of the backfill material. The same results were obtained in cases with anchors. It was found from the phase relationship between the inertia acting on the overburden and the dynamic earth pressure from the backfill material that when the inertia acting on the overburden was directed toward the ground, dynamic earth pressure from the backfill material increased. This may be because the EPS fill was pressed by the backfill material. Judging from this, the total earth pressure of the backfill material when inertia acted forward causing sliding or overturning was smaller than static earth pressure. Determining the phase relationship under irregular ground motions is a future challenge. In actual design, using the inertia of backfill material as an incremental dynamic earth pressure when seismic inertial acts forward is considered on the safe side.

4. Aseismic design

4.1 Aseismic design method[12)]

Numerous points need to be identified as to the seismic behavior of EPS fill and its contribution to the stability of the whole structural system when residual deformation occurs. Study of aseismic performance of EPS fill has recently been requested when the overturning of EPS fill is expected to have a serious impact on adjacent structures or require a long time for restoration and thus considerably undermine the functions of the structures. How much detailed study should be made of aseismic performance is determined by the location of the EPS fill or the portion of a structure where EPS fill is applied.

A flow of aseismic design of EPS fill related to dynamic analysis is outlined in Figure 12. As the first step of aseismic design, the response acceleration of the overburden such as the pavement or concrete slab located at the gravity center of the EPS fill, which is generally top-heavy, is estimated by a simple or detailed method. The response acceleration is converted to seismic intensity and applied to various portions of the EPS fill. Then the stability of the EPS fill against sliding and overturning and its bearing capacity are checked by a modified seismic coefficient method. Described below are the response acceleration estimation method and checking of stability during an earthquake.

4.2 Study procedure

(1) Estimation of response acceleration

Response acceleration of the overburden of EPS fill such as the pavement or concrete slab can be obtained by either of the following two methods.

1) Simple method

The mass of EPS fill is much smaller than that of the overburden such as the pavement or concrete slab. Then, the EPS fill is represented by an elastic beam for which mass can be ignored, and resonance frequency is calculated by the following equation.

When the back of the EPS fill is sloped or graded, an equivalent model of the slope is made (Figure 13). Then, the response acceleration during an earthquake can simply be estimated from the resonance frequency obtained as described above, acceleration response spectrum according to the soil class, and the damping characteristics of the entire EPS fill obtained by the shaking test shown in Figure 14.

2) Detailed method

Response accelerations of various portions of EPS fill are obtained by dynamic response analysis (a finite element method). Existing studies have confirmed that the stiffness of EPS hardly decreases corresponding to shear strain. EPS is therefore regarded as a linear elastic body with a certain level of damping. In view of the damping owing to friction between blocks, a damping factor (which varies according to the input ground motion) larger than that of the material can be used as an apparent initial damping factor of the EPS fill. Analysis methods include the response spectrum method, which uses input acceleration spectra of ground motions, and the time history response analysis, which uses directly input seismic waves.

(2) Checking of seismic stability

Estimated response accelerations are converted to seismic intensities and applied to various portions of EPS fill. Then, the stability of the EPS fill against sliding and overturning, and its bearing capacity are checked by a seismic coefficient method (modified seismic coefficient method).

1) Stability against overturning

Resistance owing to friction on the bottom surface of EPS fill should exceed the lateral force calculated from inertia forces at various portions of the EPS fill (Figure 15).

2) Stability against overturning and bearing capacity

Resisting moment around the toe of the EPS fill under the self weights of the fill and the overburden should exceed the acting moment calculated from inertia forces of various portions of the fill. Soil resistance balancing the above forces should be smaller than the ultimate bearing capacity of soil.

Shown below are considerations while checking the stability.

(i) Earth pressures from backfill material

Earth pressures from backfill material are calculated as inertia forces of backfill material when the width of the backfill is small relative to the height at a given stage.

(ii) Effects of metal connectors

Metal connectors prevent sliding between blocks even during strong shaking. It can therefore be assumed during design that EPS blocks behave as a united body unless blocks have an extremely small width.

(iii) Effects of anchors

When designing EPS fill, anchors are used to carry the excessive load not carried by the fill.

4.3 Design method

(1) Aseismic design method when resisting structures exist in front of EPS fill

When structures such as piles and H-steels in front of the EPS fill resist the deformation of the fill, forces acting from the EPS fill on the resisting structures are considered during design. For lateral forces, the variance between the acting force calculated from inertia forces of various portions of the EPS fill on the assumption of an ultimate state without front structures, and the resisting force on the bottom surface of the EPS fill due to friction is assumed to act on the structures in front of the EPS fill. Bending moment is considered in the same manner. Additionally, inertia forces acting on the structures in front of the EPS fill are considered when checking the stability against sliding and overturning, bearing capacity, material strength and deformation.

(2) Aseismic design methods applied to EPS fill in weak soils

The above aseismic design method is basically applicable to EPS fill in weak soils. The following points, however, needs to be taken into consideration (Figure 19).

1) Amplification of ground motions

Amplification of ground motions in weak soils needs to be considered when estimating response accelerations. Specifically, amplification of ground motions in weak soils is considered using the acceleration response spectra and input ground motions determined for respective soil classes based on the records of strong shaking for various soil classes specified in the Specifications for Highway Bridges: Part V, Seismic Design[13).]

2) Stability against slip failure of EPS fill involving the soil

In weak soils, earthquake-induced inertia forces may cause failure of the EPS fill involving the soil. Stability against slip failure involving the soil needs to be checked by analyzing the stability against rotational slip considering inertia forces.

3) Stability against uplift

When the bottom surface of the EPS fill is in the saturated sand where liquefaction is expected, stability against uplift due to excessive pore water pressures needs to be checked.

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Table 1 Recent great earthquakes and EPS fill near their focus

Year/month	Earthquake	Affected area	JMA(**)	Maximum acceleration (gal)	Maximum displacement (cm)
1993. 1	1993 Kushiro-Oki Earthquake	Kushiro	6	920	Horizontal 11
1993. 2	1993 Noto-Hanto-Oki Earthquake	Wajima	5	130	
1993. 7	1993 Hokkaido-Nansei-Oki Earthquake(*)	Okusiri	6	298	
1994. 10	1994 Hokkaido-Toho-Oki Earthquake	Kushiro Nemuro	6 5	353	
1994. 12	1994 Sanriku-Haruka-Oki Earthquake	Hachinohe	6	604	Horizontal 4
1995. 1	1995 Hyogo-Ken Nanbu Earthquake	Kobe	7	1, 764	Horizontal 18 Vertical 10

* A maximum acceleration of 1576 gal was observed in the aftershock.
** The Japan Meteorological Agency Intensity Scale.

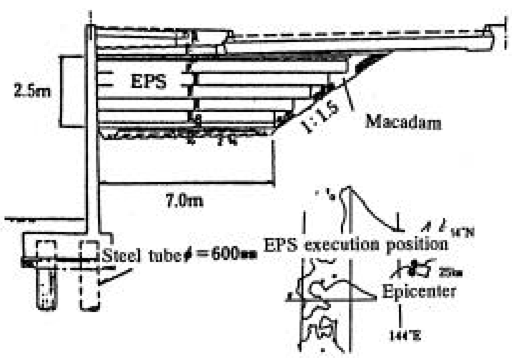


Fig.1 Profile of EPS fill and the focus (Betsuho district in Kushiro-cho)

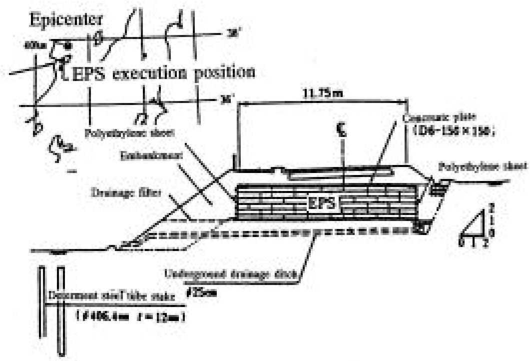


Fig.2 Profile of EPS fill and the focus (Semmaida district in Wajima City)

Fig.3 Epicenter position of the Hyogo-Ken-Nanbu EQ. and also an execution position of EPS structure

Table 2 Damages to EPS embankments due to the 1995 Hyogo-Ken-Nanbu earthquake

No.	Date of construction	Owner	Work	type	Construction volume (m^3)	Damage due to earthquake
1	1991.01	Kobe City	Back-filling of steps of Rokko Island underground parking lot	D-16	330	Hair cracks on the road surface were identified with no change observed in the entire EPS embankment.
2	1991.09	Kobe City	Construction of No.1 parking lot at Harborland	D-16	120	No deformation observed.
3	1992.07	JR West Japan	Construction of new No.2 platform at JR Amagasaki Station	D-16	620	No deformation observed.
4	1992.12	Kobe City	Tarumi sewerage works (Second stage)	D-16	2,440	No deformation observed.
5	1993.03	Hankyu Railways Nose Railways	Construction of new platform at Kawanishi Station	D-16	331	No deformation observed.
6	1993.03	JR West Japan	Improvement work of platform at JR Amagasaki Station	D-16	1,052	No deformation observed.
7	1994.01	JR West Japan	Construction of new No.4 platform at JR Amagasaki Station	D-16	175	No deformation observed.
8	1994.06	Private sector	Construction of Ark Yokohama Golf Club facility	D-16	1,001	No deformation observed.
9	1994.07	Kobe City	Embankment attached to Takamatsu bridge abutment	D-16	66	Due to the liquifaction of the steel pipe foundation, the embankment was titled, and the filled sand under EPS was washed away by river stream, resulting in the subsidence of EPS embankment (10cm).

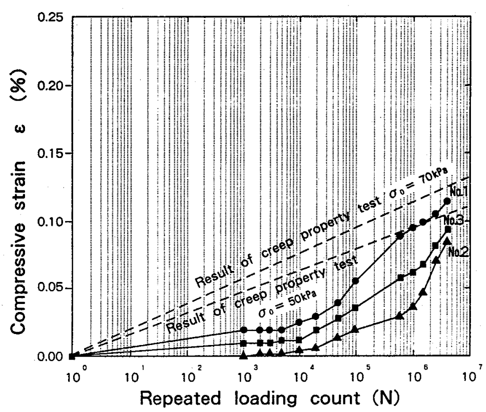


Fig. 4 Frequency of cyclic loading and compressive strain

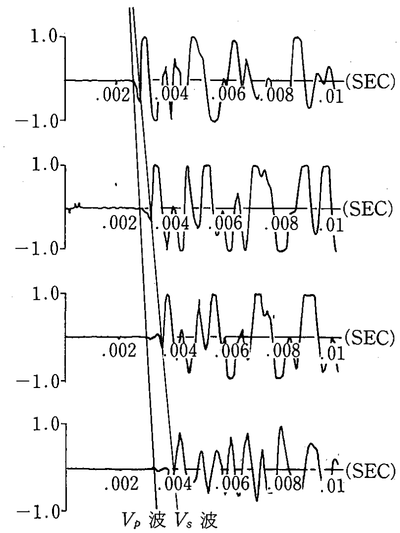
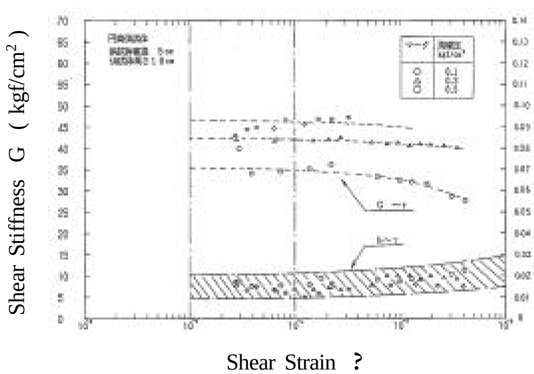


Fig. 5 Sample result of a model shake-table test



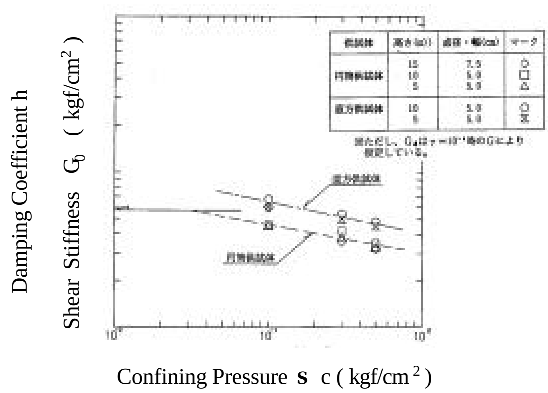


Fig. 6 Shear stiffness, damping coefficient and shear strain Fig.7 Shear stiffness Go and confining pressure

Table 3 Objectives of test models and test details

Model	Purpose	Description	
1	To elucidate the fundamental dynamic characteristics of an EPS block and the effect of a connecting metal	Two ways with and without connecting metal and another two ways with defferent overburden loads	
2	To elucidate the effects of embankment gradient on the dynamic characteristics and stability of EPS (using a half-scale model).	Embankment gradient	Two ways with different overburden loads
3		Embankment gradient	
4		Embankment gradient	
5	To elucidate the effects of back-filling soil amount on the characteristics of earth pressure as well as the effects of embankment rigidity on the tension of anchors (using a full-scale model)	Without back-filling; three ways with different embankment rigidities.	
6		With a small amount of back-filling soil	Two ways with and without anchors
7		With a large amount of back-filling soil	

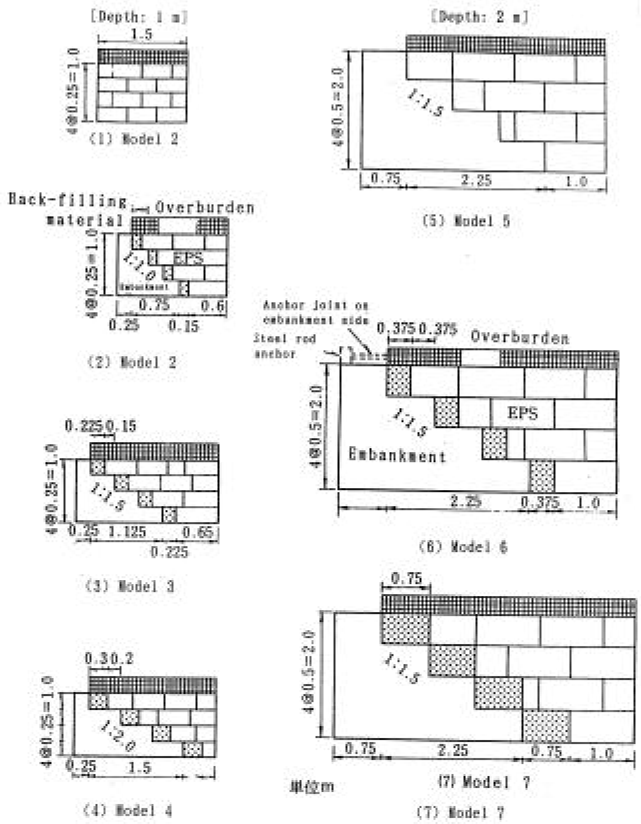


Fig8 Test models (profiles)

Table 4 Shaking conditions

Step	Frequency of input sine wave	Duration	Maximum amplitude
1	1 to 20Hz	Until steady response	Approx. 50 gal
2	Resonant frequency of model	40 waves	50 to 200 gal
3	Model 1: Resonant frequency Models 2-4 : 5 Hz Models 5-7 : 2 Hz	40 waves	200 to 1000gal

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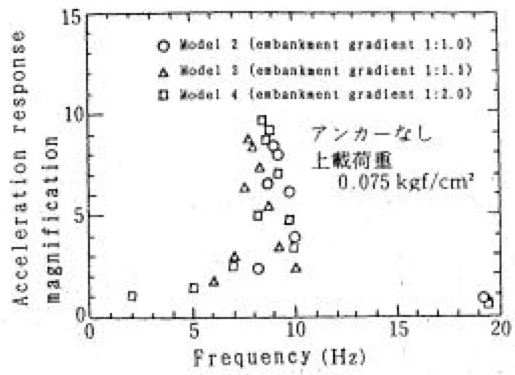

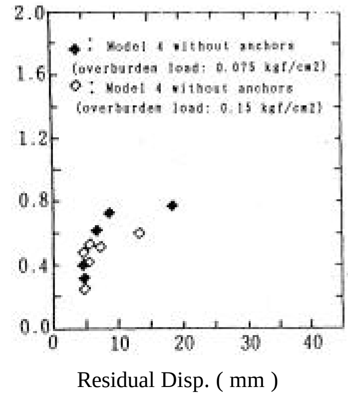

Fig.9 Effects of soil grade on frequency response Characteristics

Fig.10 Back-calculated friction factor and degree of sliding for model 1 when vertical inertia was considered

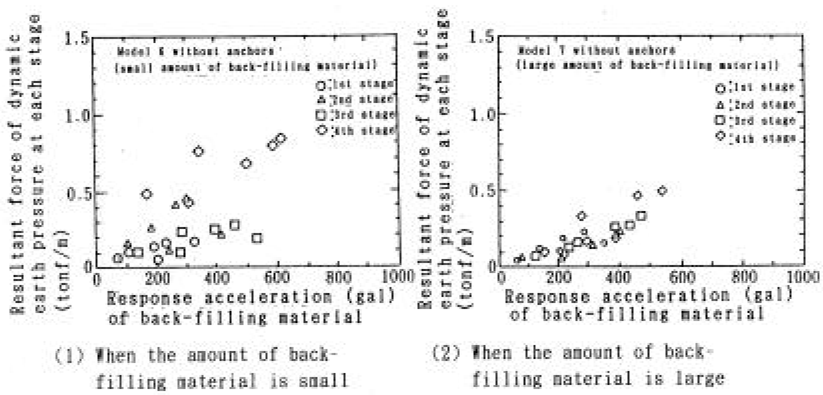

Fig. 11 Effects of the volume of backfill on dynamic earth pressure

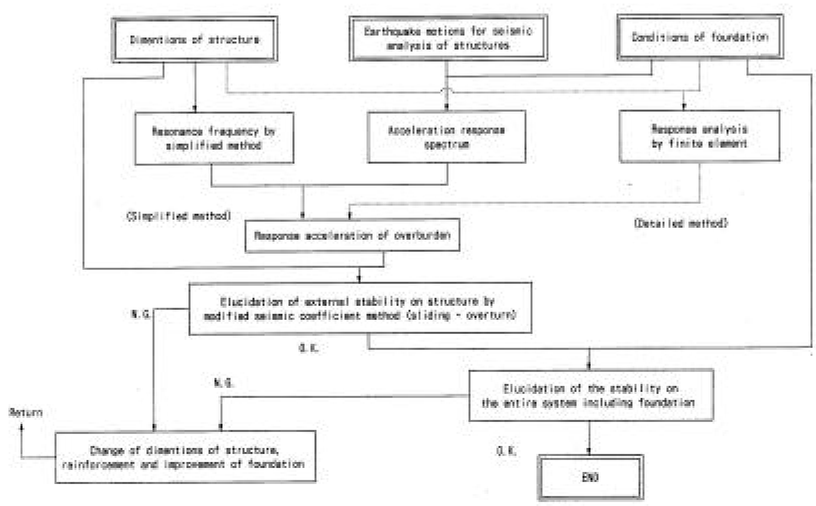

Fig.12 Flow of aseismic design of EPS fill

Calculation of the natural period of an embankment formed by EPS

Natural period: T

$$T = 2\pi \cdot \sqrt{\frac{W \cdot H}{E \cdot a \cdot b \cdot g}\left\{4\left(\frac{H}{a}\right)^2 + 1 + \frac{12}{5}(1+n)\right\}}$$

where

W is surcharge;

E and ? are the modulus of elasticity and the Poisson's ratio of EPS, respectively;

g is gravity acceleration;

H is the height of the embankment formed by EPS; and

a and b are the length and the width of the structure, respectively.

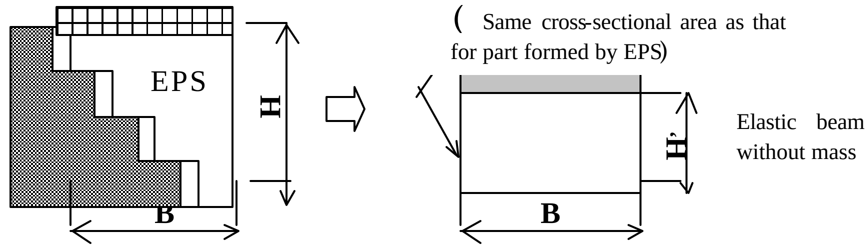

Fig.13 Equivalent model for slope section created by a simplified method

(EPS Development Organization: Method –Super Lightweight Banking Using Expanded Poly-styrol(EPS), Rikoh Tosho Co.,Ltd.))

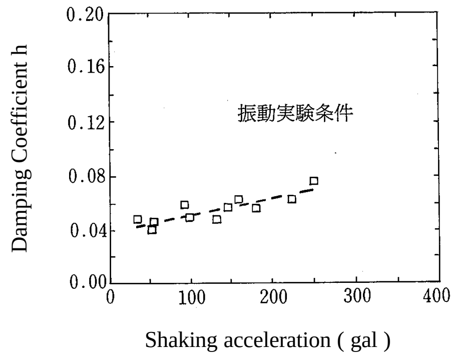

Fig.14 Damping factor of EPS fill

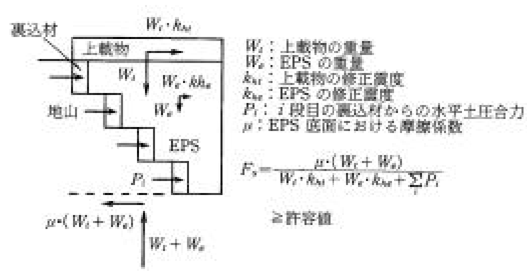

Fig.15 Stability against sliding

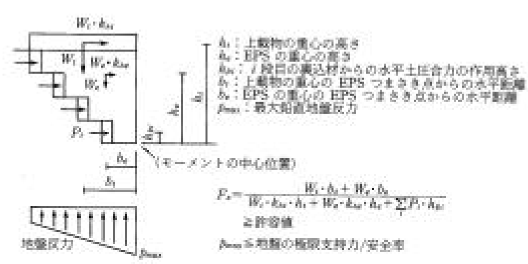

Fig.16 Stability against overturning and bearing capacity

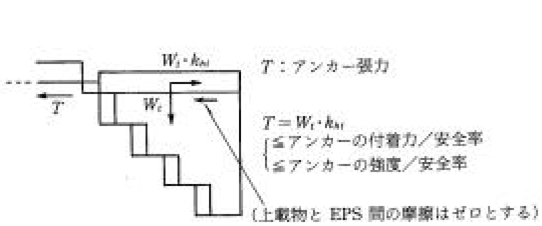

Fig.17 EPS fill with anchors

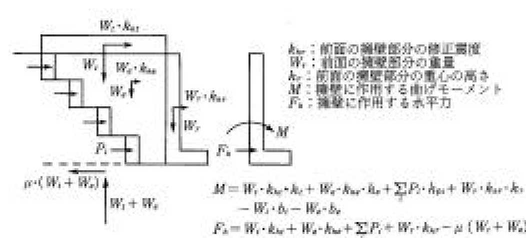

Fig.18 EPS fill with resisting structures in front of the fill

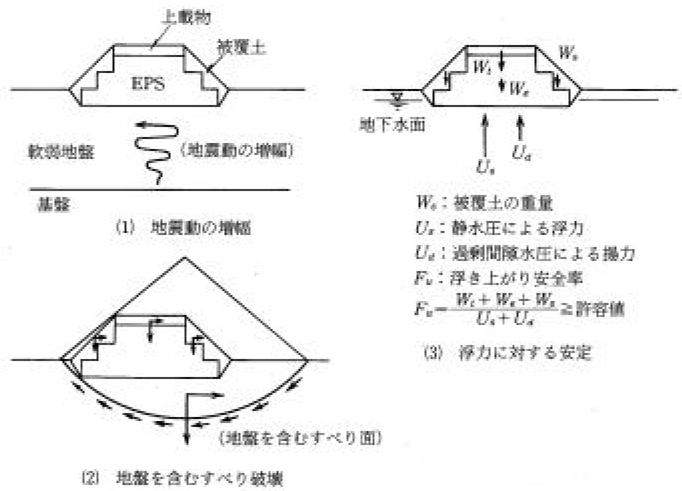

Fig.19 Aseismic design methods applied to EP S fill in weak soils