

APPENDIX F

ANALYSIS OF MAYSVILLE BRIDGE FOR EARTHQUAKES

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NCHRP 12-48

DETERMINISTIC SEISMIC ANALYSIS OF THE MAYSVILLE BRIDGE

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Date: 2000 March 28

Introduction

The object of this report is to present the results of the deterministic seismic analysis performed on the Maysville Bridge.

Description of the Maysville Bridge

The Maysville Bridge carries route US 62/68 traffic over the Ohio River between Kentucky and Ohio. Only the two main piers of the Maysville Bridge are subject to vessel collision forces, all other piers are located out of the waterway. The two main piers support cable-stayed spans of 400'-1050'-400'. Each of the main piers is supported on 16 cast-in-place concrete piles with a 6' diameter in a 4X4 arrangement. The piles are socketed into a hard bedrock approximately 34' below the pile cap. A sand layer of varying thickness overlies the bedrock.

Response Spectra

The response spectrum used in the analyses was developed from the standard spectrum provided in AASHTO [Ref 1], which defines the elastic seismic response coefficient, C_{smr} , as

$$\frac{1.2AS}{T_m^{2/3}} \leq 2.5A \quad \text{for } T < 4 \text{ sec} \quad (\text{Equation 1})$$

$$= \frac{3AS}{T_m^{4/3}} \quad \text{for } T > 4 \text{ sec} \quad (\text{Equation 2})$$

where

T_m = period of vibration of the m^{th} mode (in seconds)

A = acceleration coefficient

S = site coefficient

The above equations are based on 5% damping.

Soil profile I (applicable for rock and stiff soils) is assumed as per the design drawings. This gives $S=1.0$. The normalized seismic response curve (where $A=1.0$) for soil profile I is presented in Figure 1.

The acceleration coefficient, A , for the Maysville area was determined from peak ground acceleration (PGA) contour maps and tables provided by USGS [Ref 2] for three separate uniform risk models:

1. Probability of exceedance during a 50-year period of 10%
2. Probability of exceedance during a 50-year period of 5%
3. Probability of exceedance during a 50-year period of 2%

These represent the 500, 1000 and 2500-year return events. The 500-year return event represents the design earthquake, whereas the 2500-year return event represents the maximum credible earthquake.

A summary of the peak ground accelerations at Maysville for the various return periods is shown in Table 1.

Return Period (years)	Probability of Exceedance in 50 yrs	Peak Ground Acceleration
500	10%	3.9%g
1000	5%	6.8%g
2500	2%	13.2%g

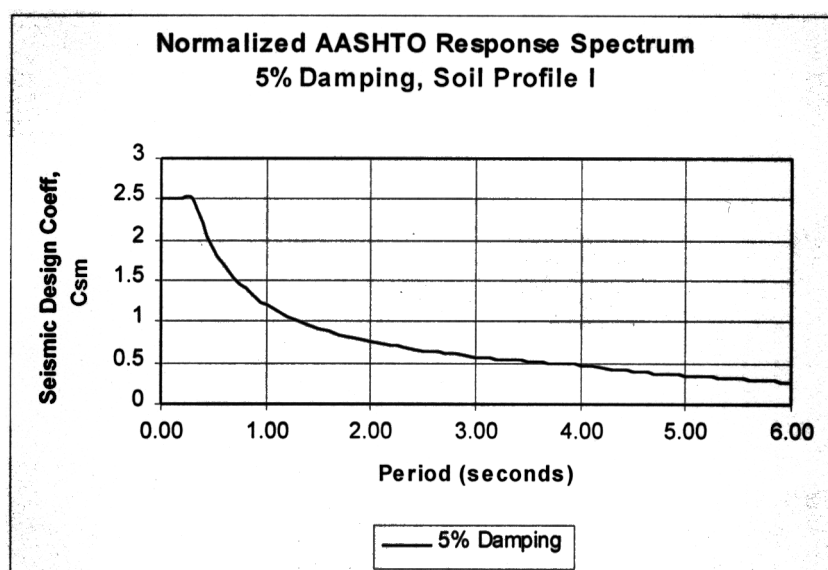
Table

Response spectra developed based on the above peak ground acceleration values and the AASHTO model (equations 1 and 2) are presented in Figures 2-4 for the 500, 1000 and 2500 year return events. These are based on 5% damping and marked accordingly.

For a structure such as the Maysville bridge, a damping ratio of 2% was considered more appropriate, than the 5% on which the above spectra are based. The response spectra were therefore scaled up accordingly, using the Newmark/Hall method [Ref. 3]. With this method, the three legs of the response spectrum

(representing the approximately constant amplifications of acceleration, velocity and displacement response) are scaled by the ratio of amplification factors for the two levels of damping. It was assumed that the AASHTO response spectrum is a mean plus one standard deviation (84th percentile) spectrum. Accordingly, the ratios for modifying a 5% damping spectrum to a 2% damping spectrum for the three legs were computed as 1.35 for the acceleration leg ($0.125 < T < 0.5$ sec), 1.27 for the velocity leg ($0.5 < T < 3.33$ sec), and 1.20 for the displacement leg ($T > 3.33$ sec). For simplicity, a constant ratio of 1.35, based on acceleration response, was applied to the entire spectrum.

The resulting spectra are also shown on Figures 2-4, marked 2% damping. To summarize, these spectra are based on the AASHTO model (equations 1 and 2), the acceleration coefficients in Table 1, a site coefficient of 1.0, and a factor of 1.35 to modify the derived spectra from one based on 5% damping to one based on 2% damping. No response modification factors were applied.



Figure

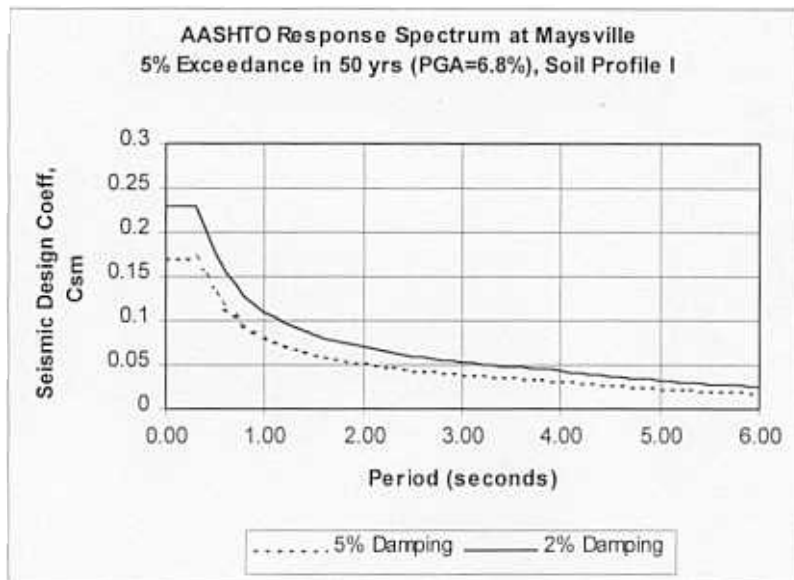


Figure 2

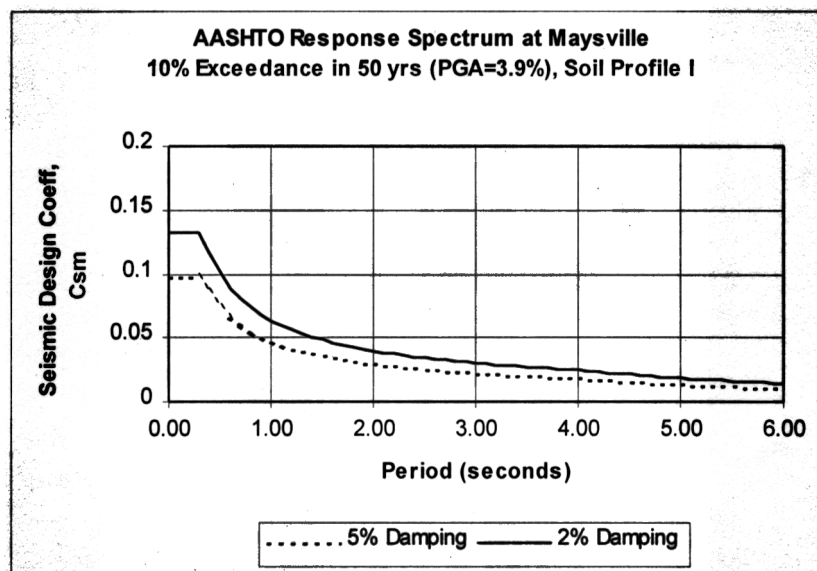


Figure 3

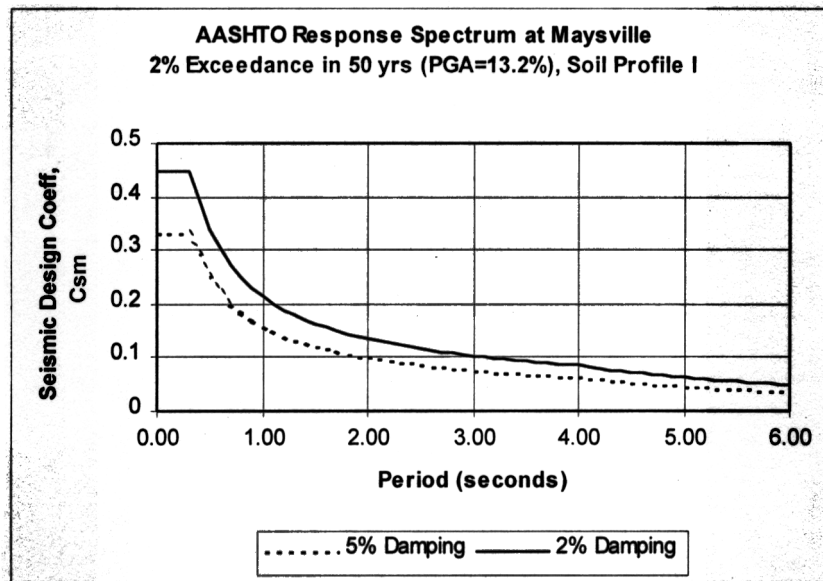


Figure 4

Soil Springs

Representative foundation spring stiffnesses were developed for five cases: 0%, 25%, 50%, 75% and 100% scour of the top sand layers. The foundation at each pier consists of 16 drilled piles spaced 12' apart in the bridge longitudinal direction and 18' apart in the bridge transverse direction (Figure 5). The pile group stiffness was derived by analysis with the program GROUP [Ref. 4]. Pile head elevation was taken at EL 447, i.e., bottom of the foundation seal. The analyses were based on the soil profiles at Pier 6, since the profile at this pier indicates top of rock at a lower elevation, compared to Pier 5, thus providing more potential scour depth for the investigation. While the design profile indicated ground elevation at EL 437, the profile was modified slightly by addition of a layer of loose sand to EL 447. This was considered the base case, from which increasing layers of soil were removed to develop the foundation springs for the four scour cases.

Profiles analyzed for the five levels of scour (none, 25%, 50%, 75% and 00% scour) are shown in Figure 6.

Pile group stiffness was derived for all five cases using the soil properties provided on the design drawings and summarized in Table 2. Forces applied to the pile head were based on earthquake load levels for the 500-year return period obtained from a fixed base analysis of the bridge.

EL	Layer	γ (pci)	ϕ (deg)	c_u (psi)	k_s (pci)
To 447	Loose Sand	.067	28		20
To 437	Medium Sand	.069	32		60
To 425	Boulder	.072	38		125
To 413	Limestone	.075		3,472	38,773

Table 2

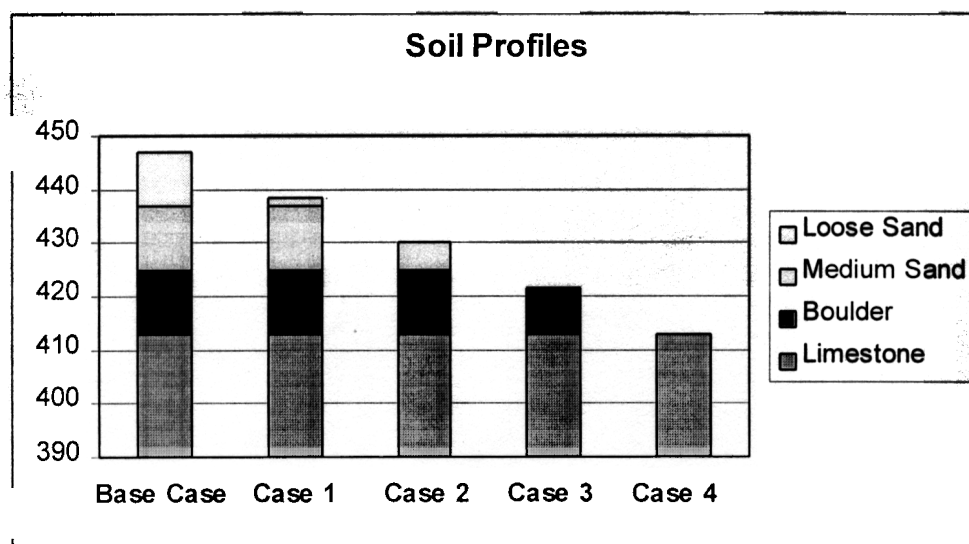


Figure 6

Results of the pile group analyses show that the foundation essentially behaves linearly at this load level. Vertical, lateral and rotational soil springs representing the stiffness at the pile head are summarized in Table 3. Variation of lateral stiffness with scour depth is shown graphically in Figure 7 to illustrate the impact of scour on foundation stiffness.

Percent Scour	Remaining Soil Depth Above Bedrock (ft)	Vertical Kv (kip/ft)	Long. Kx (kip/ft)	Transverse Ky (kip/ft)	Rotat. Krx (kip-ft/rad)	Rotat. Kry (kip-ft/rad)
0%	34.0	2.79E+05	2.17E+05	2.25E+05	3.11E+09	1.40E+09
25%	25.5	2.80E+05	1.66E+05	1.68E+05	3.13E+09	1.40E+09
50%	17.0	2.80E+05	1.32E+05	1.34E+05	3.21E+09	1.43E+09
75%	8.5	2.80E+05	1.25E+05	1.27E+05	3.35E+09	1.49E+09
100%	0.0	~2.79E+05	~1.25E+05	~1.27E+05	~3.35E+09	~1.49E+09

Table 3

Effect of Scour on Foundation Lateral Stiffness

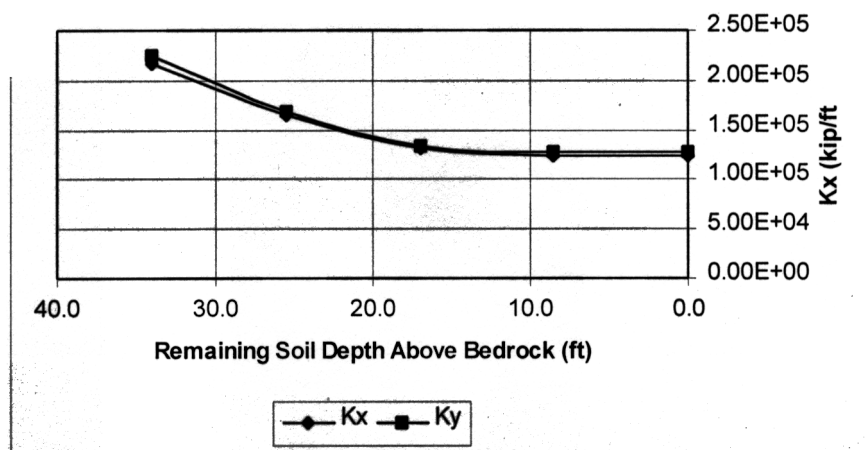


Figure 7

Figure 7 illustrates that the effect of scour is significant in the top layer, but as the remaining soil depth decreases, the effect of scour on lateral stiffness reduces substantially. It is thus predicted that the period of the structure, and thus the seismic load level, will vary somewhat between the base case and cases 1 and 2 (0, 25 and 50% scour), but not noticeably between cases 2 and 3 (50 and 75% scour) and nil between cases 3 and 4 (75% and 100% scour).

The appropriateness of these soil springs was verified by applying the forces obtained from the seismic analysis using these foundation springs to the pile head and deriving new foundation springs from a second iteration of GROUP analyses (this time also increasing the force level to that of the 2500-year return period earthquake). Revised soil spring stiffnesses were not significantly different from the first pass stiffnesses, and thus were deemed acceptable. This verification was carried out only for the base case.

Earthquake Response Spectrum Analysis

The seismic analyses were carried out on the Maysville Bridge model developed during a previous construction engineering contract. The model represents the bridge at the time it is open for traffic and is thus suitable for the current purpose. The model has 660 nodes, 527 members and 80 cable members. Fixed base conditions at the bottom of piers 5 and 6 were removed and replaced with the soil springs derived above and summarized in Table 3. Since the soil springs derived for the 75% and 100% scour cases were substantially the same, it was only necessary to analyze four, rather than five cases.

Eigenvalue solutions were carried out for all four cases. 99 modes were pulled, resulting in approximately 100% mass participation in the lateral directions and 95% in the vertical direction. The first 10 modes are summarized in Table 4.

Mode	Period (sec)	Description
1	5.448	First longitudinal mode, of the deck only. This mode does not actually occur as the locking mechanism between deck and towers will activate during seismic loading.
2	2.854	Vertical.
3	2.198	First longitudinal mode of entire bridge structure.
4	2.046	First transverse mode.
5	1.667	Local transverse mode.
6	1.643	Transverse mode.
7	1.446	Local vertical mode.
8	1.221	Longitudinal mode.
9	1.168	Vertical mode.
10	1.165	Torsion of deck

Table 4

Seismic forces were obtained by applying the response spectra presented in Figures 2 through 4 and summing modal contributions using the CQC procedure. Member internal forces were then combined as follows for each case:

$$1.0DL \pm \{1.0EQ_x + 0.3EQ_y\}$$

$$1.0DL \pm \{0.3EQ_x + 1.0EQ_y\}$$

where DL = dead load

EQ_x = seismic load in the bridge longitudinal direction

EQ_y = seismic load in the bridge transverse direction

This procedure was carried out for the base case, the results of which confirmed that the member loads for the different earthquake levels can be obtained by

linearly scaling the results from the analysis using a single spectrum. This is possible since the individual spectra were derived by scaling the AASHTO response spectrum, based on 10% probability of exceedance in 50 years and a 5% damping ratio. Thus, for each scour model the earthquake analysis was carried out for only one spectrum and the results were then scaled linearly to obtain the member loads for all spectra.

The maximum values obtained from these combinations are presented in Figure 8 (a-d) for Pier 6. Values for Pier 5 are similar and therefore not shown. The results indicate that the foundation stiffness and overall seismic response of this particular structure are not affected significantly by varying scour levels. The foundation lateral capacity was not exceeded for any of the earthquake levels. It was determined that a peak ground acceleration of 24.6% would be required to reach the foundation lateral capacity (yielding of piles in flexure) of 8000 kips. This level of ground acceleration is estimated to represent a return period of approximately 5900 years.

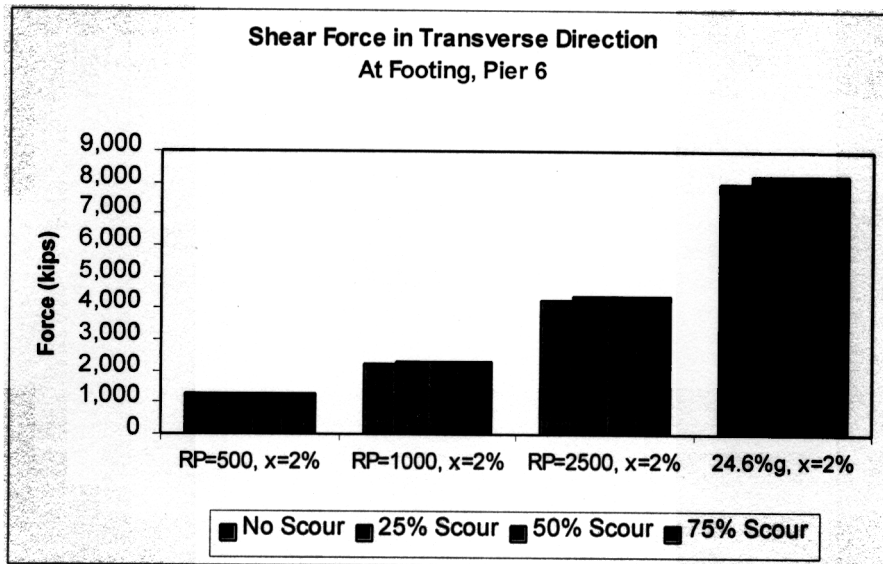


Figure 8a

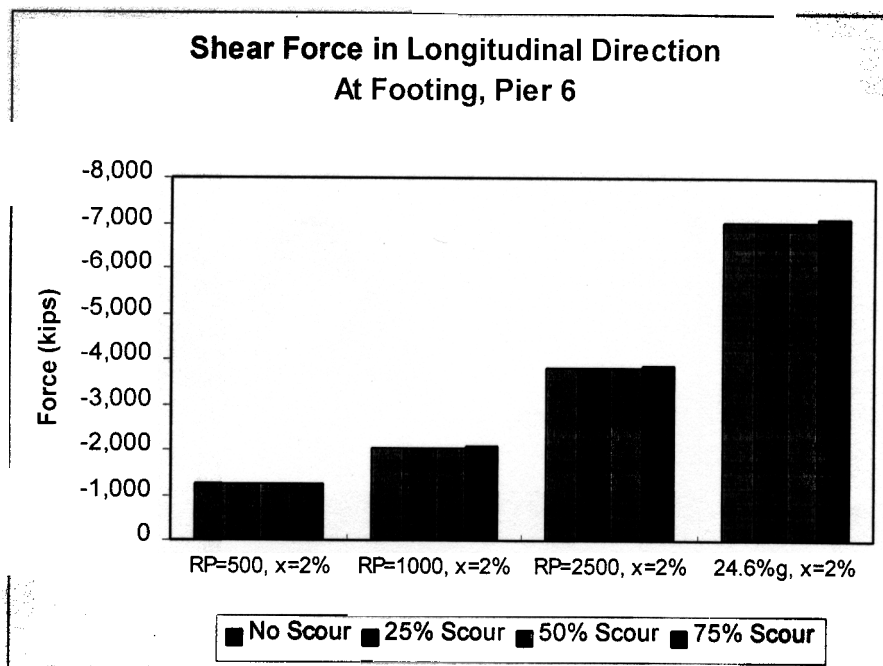


Figure 8b

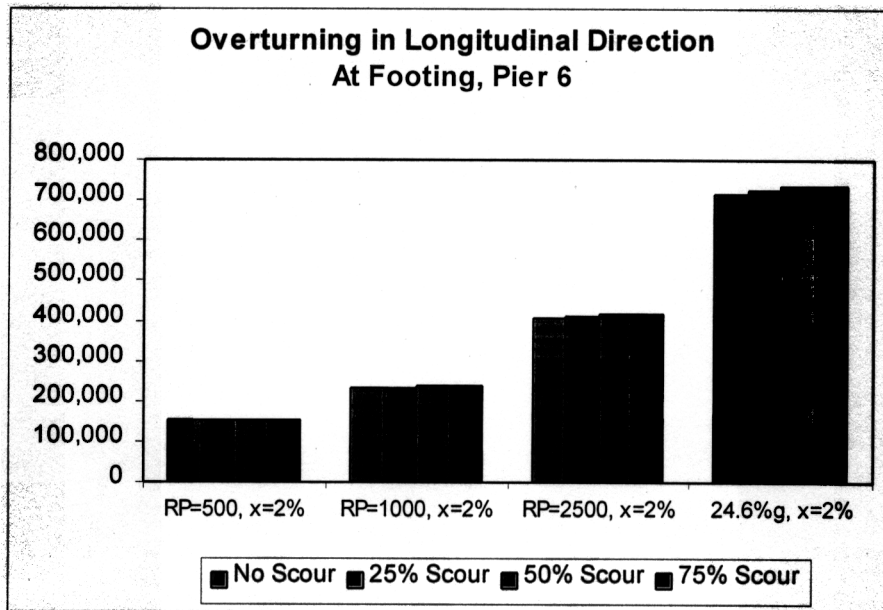


Figure 8c

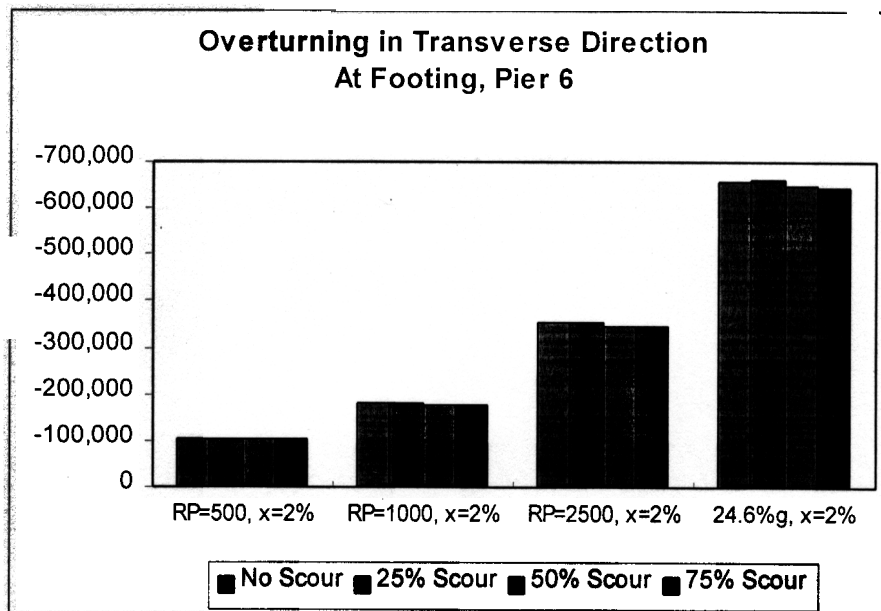


Figure 8d

References

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