Report of Trial Design #2

as submitted by the SEI-BPAD Design Practices Committee

Background: The Design Practices Committee (DPC) of the SEI Business and Professional Activities Division (BPAD) has completed its second trial design study. This exercise was part of an ongoing program designed to test how practicing engineers actually apply codes and standards. The exercise consisted of two problems described below. These problems were solved by respondents solicited through STRUCTURE magazine, the SEI web page, and personal contact by committee members. Solutions were collected by the committee, reviewed and the results tabulated.

The DPC received 19 solutions of the Pile Cap Problem and 22 solutions to the Shearwall Problem. The average experience level of the participants was over 12 years. More than one half of these participants have graduate degrees and nearly all were engineers who work in design offices.

Results of Pile Cap Design Problem: This problem, as originally conceived, was to include a sketch showing physical dimensions of the pile cap and the piling arrangement. Unfortunately (or fortunately), the sketch was not sent out when the problem was placed on the web site, providing the committee with some interesting solutions.

In general, the size of the column base plates designed by the respondents could be grouped into two general categories, with plan dimensions of 32x35 inches and 24x24 inches, depending on whether the respondent took into account the increase in bearing stress allowed by the American Institute of Steel Construction (AISC) Code due to the ratio of base plate area to pile cap area. The associated base plate thicknesses varied from 2¼" to 4".

The size and number of anchor rods, as well as the weld between the column and base plate, were generally based on experience, office practice or what "looked right," since no design calculations were required. Nearly all respondents specified a 5/16" fillet weld, although two answers called for 1-inch and 2-inch fillet welds. This facet of the exercise was meant more to reveal typical office practice than understanding of a code provision. The DPC's opinion was that there is no code provision requiring a minimum size anchor rod. In cases where there is minimal load to be transferred, weld size is controlled by AISC minimum requirements.

In reviewing the answers the DPC determined that all respondents correctly calculated the ultimate load (Pu) for concrete, with differences in the answers attributed to whether or not the respondent considered the weight of the pile cap.

Calculated pile cap thickness varied from 39" to 84". The control solution prepared by the DPC had assumed a thickness of 54". The distance from the bottom of the pile cap to the reinforcing steel varied from 3" (sitting directly on the piles) to 10" (adding the 3" clearance above the top of the pile). Both solutions are common throughout the industry. The wide variation in thickness and effective depth produced associated variations in the amount of reinforcing steel required, with the shallower pile caps often containing lighter reinforcing. The DPC did not find a correlation between calculated pile cap thickness and the experience level of the practitioner, which was surprising. The DPC expected the more experienced engineers to select a thickness in the 50" to 56" range, which was not necessarily the case.

The wide variation in shear capacity (Vc) was the result of the designer concluding that one-way or two-way deep beam action applied or whether punching shear controlled. The punching shear capacities varied from 2408 kips to 3027 kips. Beam shear capacities varied from 485 kips to 866 kips. This response is not surprising, since several factors would lead us to believe that this would be the case: The extreme variation in pile cap thickness would presuppose a wide variation in shear capacity which is a function of concrete thickness. Also, ACI 318 doesn't fully address appropriate design of pile caps, creating confusion among designers. Many engineers rely on the CRSI procedure to design pile caps.

Nearly all respondents assumed the customary pile spacings of 3'-0", although one answered with a 2"-0" spacing and another with a 6'-0" spacing. Since pile spacing directly affects the bending moments in the pile cap, it was understandable that variations in reinforcing quantities would be considerable.

Results of Shearwall Design Problem: Of the 22 engineers who submitted solutions, 21 completed the entire problem. One respondent questioned the constructability of a 12-inch wall, and computed only the design loads. Five of the respondents were from states located in Seismic Zone 0 where the Uniform Building Code (UBC) is not adopted. The other 17 solutions were from respondents in states which use the UBC and are located in Seismic Zones 3 or 4. There were large variations in solutions to the problem. The solutions from the respondents who were less familiar with seismic design did differ significantly from the other solutions.

The calculated base shear for the building varied between 1486 kips and 2707 kips (a difference of 82%), with an average of 1927 kips. This large variation was due to different assumptions regarding the overstrength and ductility factor, R, the calculated seismic dead load of the building, and interpretation of the

TABLE OF RESULTS PILE CAP PROBLEM

Respondent	P_u (kips) (Concrete)	Pile Cap Height [illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1	2564	53"	43"	(21) - #8	2632	3'
2	2564	47"	38"	(11) - #10	2749	3'
3	2564	53"	45"	(12) - #9	2758	3'
4	2560	57"	47"	(13) - #8	2880	3'
5	2576	39"	32"	(14) - #9	291	3'
6	2564	42"	38.5"	(15) - #8	855	(?)
7	2564	84"	80.4"	(13) - #9	1246	3'
8	2648	53"	45"	(14) - #10	2516	3'
9	2565	42"	38"	(35) - #6	1038	6'
10	2565	50"	39"	(13) - #8	2138	3'
11	2625	52"	44"	(15) - #9	3027	3'
12	2564	53"	44.73"	(13) - #10	2635	3'
13	2564	52"	42"	(13) - #9	2683	3'
14	2564	72"	68.5"	(48) - #9	866	4'
15	2564	84"	79"	(9) - #18	866	3'
16	2649	54"	46"	(12) - #10	2793	3'
17	2564	54"	46"	(12) - #9	2769	3'
18	2564	60"	54"	(20) - #8	2408	3'
19	2564	43"	40"	(13) - #10	485	2'

base shear equations 30-4 and 30-5 from Chapter 16 of the UBC. Four respondents assumed this building was a bearing wall structure with R = 4.5. The other 18 respondents assumed it was a frame building with R = 5.5. The calculated seismic dead load of the building varied between 9904 kips and 14,268 kips (a difference of 44%), with an average of 11,847 kips. Three respondents used equation 30-4 to calculate the seismic base shear. Equation 30-5, which the other 19 respondents used, gives a 20% lower design base shear.

The unfactored base shear, for the wall which was to be designed, varied between 794 kips and 1737 kips (a difference of 119%), with an average of 1103 kips. Three respondents ignored torsion on the building, while two others made significant arithmetic errors in determining the effects of torsion.

The unfactored overturning moment for the wall varied between 10,655 kip feet and 64,360 kip feet (a difference of 504%), with an average of 45,112 kip feet. The calculated overturning moments were more or less proportional to the calculated base shears, except for two respondents who made significant arithmetic errors in determining the overturning moment.

The reduction/reliability factor, p, varied between 1.00 and 1.50, with an average of 1.26. Five respondents ignored this factor altogether.

Calculated gravity loads for the wall varied between 414 and 1170 kips (a difference of 183%), with an average of 678 kips. Five respondents ignored gravity loads altogether, and a sixth respondent ignored all gravity loads except the dead weight of the wall itself.

The calculated vertical earthquake load, Ev, varied between 30 and 159 kips (a difference of 430%), with an average of 96 kips. Ten respondents ignored this load entirely.

Load combinations for dead load + live load + seismic load were inconsistent. Five respondents used equations 12-5 and 12-6 from Chapter 16 of the UBC multiplied by 1.1, as required in Section 1612.2.1. Eleven respondents used the same equations without the 1.1 factor, based on recommendations by Dr. S. K. Ghosh. Four other respondents designed for earthquake loads only, ignoring dead load and live load, while one respondent designed for earthquake loads only, multiplied by the 1.1 factor times an additional factor of 1.3.

Calculated values for the wall design base shear varied between 889 kips and 2866 kips (a difference of 222%), with an average of 1447 kips. The calculated values for the wall design base overturning moment varied between 13,830 kip feet and 76,639 kip-feet (a difference of 454%), with an average of 54,424 kip-feet. These large variations were mostly due to an accumulation of different assumptions in the calculations leading up to these answers, rather than any one significant error.

Vertical shear reinforcing steel in the walls varied between 0.40 square inches per foot and 2.11 square inches per foot (a difference of 428%), with an average of 0.91 square inches per foot. Horizontal shear reinforcement in the shear walls varied between 0.40 square inches per foot and 3.79 square inches per foot (a variation of 848%), with an average of 1.46 square inches per foot. One respondent had a single layer of horizontal reinforcement in the wall. All other respondents did have two layers of reinforcement in each direction.

The exterior boundary reinforcement in the wall varied between 84.24 square inches and 7.90 square inches (a difference of 965%), with an average of 43.94 square inches. Seven respondents, including the five who work in non-seismic areas, did not confine the boundary reinforcing. All other respondents provided confinement. Five respondents designed confined vertical reinforcement adjacent to the window blockouts, nine other respondents showed non-confined reinforcing adjacent to these blockouts, and seven respondents showed no special reinforcing at this location.

Nine respondents designed the coupling beam with tied diagonal bars as required by code. Nine respondents added nominal reinforcement to the coupling beam, while three respondents provided no special reinforcing in the coupling beam.

The range of answers was surprisingly large, particularly from engineers who regularly design using the seismic provisions of the UBC. While several of the extreme answers were due to faulty arithmetic or a misunderstanding of certain code provisions, much of the difference was caused by accumulation of different assumptions. The solutions to this problem clearly demonstrate that capable, experienced structural engineers designing the same structural element can arrive at significantly different answers without making "errors."

Conclusion: Consistent with the results of the first Trial Design Study (ref STRUCTURE, Spring 2000 issue) the DPC has determined that there is a considerable lack of knowledge regarding requirements and applicability of the current building codes. Discrepancies in the answers presented in the Pile Cap Design problem also highlighted the fact that many engineers are too dependent on handbooks and computer programs. This theory is becoming a widely accepted fact and the trend is not easily reversed. Few offices will forego the efficiency of computer programs to allow employees to hone their skills in producing hand calculations.

The Shearwall Design Problem was unquestionably the most complicated problem the DPC has produced so far, and the results bore this out. Engineers who should have been keenly familiar with the 1997 UBC made miscalculations that resulted in a non-code-compliant structure. Based upon these results, the DPC must conclude that either the code is too complex or the practitioners were careless in its application. Both conclusions are probably valid.

Of little comfort is the speculation that structures are built every day with design errors just as egregious as the ones found in the solutions to both problems. Fortunately for the public, a combination of redundancy, overestimation of the design loading, and lack of load testing allows the structures to perform as intended instead of as designed. However, we must strive to eliminate luck, as a factor in our work, for it is sure to run out occasionally. The Trial Design process is an on-going project intended to help the structural engineering profession better understand just how good or bad we are at what we are doing.

The Design Practices Committee would like to thank all the participants who donated their time to the project. The Trial Design Process depends on qualified participants representing a broad spectrum of the practicing structural engineers. It is our long-term goal to increase participation to the extent that the results will have statistical significance. Your assistance on future exercises is of paramount importance.

NEW TRIAL DESIGN PROBLEMS

There are three new problems available: a concrete slab design problem, a steel connection design problem, and a wind/shear wall problem.

Individuals interested in participating may download the problems in electronic format from www.seinstitute.org, or send an email to: mbarter@barterse.com requesting the file(s).

Instructions on how to return the solutions are contained within the problem files.

Table of Results Shearwall Problem						
Respondent	R	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1	4.5	.192W	.183W	14,268	2707	1423
2	5.5	.211W	2.126W	11,468	2418	1330
3	5.5	.190W	.150W	12,258	1839	920
4	5.5	.190W	.150W	12,000	1800	990
5	5.5	.190W	-	11,224	2130	1618
6	4.5	-	.183W	12,447	2278	1253
7	5.5	.190W	.150W	11,027	1654	1737
11	5.5	.190W	.150W	9,904	1486	817
12	5.5	.190W	.150W	12,782	1917	1061
13	5.5	.190W	.150W	11,705	1756	1010
14	5.5	.190W	.150W	12,907	1936	1065
15	5.5	.163W	.150W	12,312	1847	1010
16	5.5	.190W	.150W	13,635	2045	1125
17	4.5	.230W	.183W	11,562	2116	1164
18	5.5	-	.150W	10,580	1587	794
19	5.5	.190W	.150W	12,094	1814	1000
20	5.5	.190W	.150W	11,561	2202	1211
21	5.5	.144W	.150W	10,605	1527	840
22	5.5	.154W	.150W	10,033	1505	752
23	4.5	.232W	.183W	11,631	2132	1173
24	5.5	-	.150W	13,000	1950	1014
25	5.5	.190W	.150W	11,623	1743	959
Average	-	.190W	.157W	11,847	1927	1103