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Modification of TS-500 Flat-Plate Punching Capacity Equation for Interior Square Columns

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Abstract

The safety of design equations given in the national building codes are usually questioned and verified with the available test data in the related field. In this study, tests on punching capacity of internal square columns are searched for and the available test data is collected into a database. The data are reproduced by the use of TS-500 design equations, and the discrepancy from the test values are highlighted. As a result, a modification factor is proposed for TS-500 flat-plate punching design equation.

Introduction

Flat slab is one of the most widely used structural systems in reinforced concrete construction due to its high degree of architectural efficiency. It uses simple formwork and reinforcing arrangements and requires the least story height (Durrani, 1994). Although the flat plate system has been preferred widely in residential buildings in many parts of the world since the beginning of the last century, its load resistance behavior is still not perfectly understood. Because of its limited rigidity in moment transfer, a flat plate-column frame is especially vulnerable to damage through lateral cyclic displacements under earthquake loading (Farhey, 1993).

Literature Review

The major concern with the flat-plates is the brittle punching failure occurs due to the transfer of shearing forces and unbalanced moments between the slab and the column.

In seismically active zones, where structures can easily be subjected to deformations beyond their elastic design limits, slab-column connections must withstand such deformations. In other words, adequate deformability, and strength requirements should be satisfied simultaneously (Ghali, 1994).

Due to lack of capacity of the slab-column connections, some flat-plate structures failed in a brittle punching mode when they were subjected to earthquakes. For example, after the 1985 Mexico City earthquake, a number of failures of flat-slab structures were reported, some in dual systems. During the 1964 Alaska earthquake, J.C. Penney building, which comprised of flat-plates and shear walls, suffered a partial collapse. Although the Holiday Inn buildings did not collapse during the 1971 San Fernando earthquake, extensive nonstructural damage was observed (Pan, 1988).

The design procedure for eccentric punching is generally formulated such that a fraction of the unbalanced moment results a non-uniform distribution of shear stresses over the punching perimeter. In the case of unbalanced loads, the punching region is confined to an area near the more heavily loaded face of the column and the two adjacent side regions show extensive torsional cracking while the area near the opposite face may show little distress (Özden, 1998). Eccentricity is defined as the unbalanced moment over the vertical load. High eccentricities may occur under earthquake loads, and the existing gravity loads over the plate highly influence the capacity. Increasing the slab gravity load and subsequent shear level at the interior connection significantly reduces the level of the lateral drift that the specimen could attain prior to failure (Robertson, 1992). The ACI Building Code design approach for the transfer of shear and unbalanced moment at an interior connection is reported as unconservative for a direct shear ratio in excess of 0.30.(ASCE-ACI Task Committee, 1974).

Numerical Approach for the Capacity Predictions

The design methods and empirical equations in most of the building design codes, for the calculation of the punching shear strength of flat-plates, are based on the results of the experimental investigations. While the theoretical capacity prediction procedures were being developed, experimental studies were also performed in order to verify these approaches and to highlight the punching behavior of flat plates (Özden, 1998).

30 flat-plate specimens were tested in Boğaziçi University Structures Laboratory between 1995 and 2000 (Özden, 1998, Ertaş, 2000). These tests showed that, the confidence level of TS-500 punching design equation decreases while the load eccentricity increases. For this purpose, a database is prepared including the available tests in the literature serving to highlight the effect of load eccentricity. The common properties of the specimens investigated are such that the failure mode of the specimens is punching; all specimens have internal square column; the slab are cast using normal strength concrete; and slabs are without holes. Finally, column width to slab depth ratios is between 1.5 and 4 for the specimens in the database. Capacity predictions of these specimens were done by using the TS-500, EC-2 and Final Draft EC-2 equations, and compared with the test results. Specimen properties and capacity predictions can be seen in Table-1 and Table-2 respectively.

TS-500 considers only the concrete tensile strength, load eccentricity and column aspect ratio in the calculation of punching strength of the flat-plate to column connection (Equation 1). On the other hand, EC-2 and Final Draft EC-2 takes into account the effect of slab reinforcement ratio additionally (Equation 2, 3).

$$P_{TS500} = \gamma \times f_{ctd} \times u_p \times d \tag{1}$$

$$P_{EC-2} = V_{sd} \times u_p / \beta \tag{2}$$

$$V_{sd} = \tau_{rd} \times (1.12 + 40 \times \rho_l) \times d \tag{3}$$

According to the test data from the past research, punching shear capacity of the flat-plates is directly related to the load eccentricity over the column dimension ratio (e/r) for the internal square columns.

The theoretical punching shear capacity values, calculated according to the existing code equations, is reported as unconservative for high e/r ratios. The discrepancy of predictions, made according to codes, is shown in Figure 1 and Figure 2. These figures highlights that the structure may seriously be damaged during earthquakes since the eccentricity becomes very high, compared to the case under gravity loads (Ertaş, 2000).

Final Draft EC-2 is important for the Turkish designers since the new EC-2 code will replace the TS-500 in a few years. Therefore, the specimen capacities in the database are calculated according to the proposed new equations in Final Draft EC-2 (Equation 4). The results are conservative according to Draft Final EC-2 (Figure 3).

$$v_{ed} = \frac{V_{un}}{u_p \times d}\left[1 + k\frac{M_{un} \times u}{V_{un} \times W}\right] \tag{4}$$

When the test results are compared with the TS-500 predicted shear capacities, it is observed that the confidence level decreases with increasing e/r ratio. The predicted capacity becomes unconservative, after an e/r ratio bigger than 1.0 or 1.2. This region can be defined as a transition zone and it is observed that the confidence level decreases by 20-30 percent drastically after this region. After this drop, the level of confidence stay approximately constant even if the e/r ratio increases. As a result, TS-500 needs to be revised for a reduced punching capacity design. Under these conditions, a simple coefficient was proposed. The proposed capacity multiplier for TS-500 is 0.75 if e/r ratio is greater than 1.20. The initial sum of square of errors (SSE_i) for the database is 2.47, and the final sum of square of errors (SSE_f) is reduced to 1.97 after modification.

Conclusions

Using the results of 46 different test specimens from literature it was shown that TS-500 and EC-2 punching shear capacity predictions for flat plates are unconservative at high e/r ratios for inner square columns. Some of the structures that designed according to these codes may have a certain level of risk under earthquake loads. The safety of Final Draft EC-2 is observed to be adequate according to the available data set. The proposed modification factor for TS-500 equation is very simple for the designers, but to define real behavior, there should be more data considering different effects such as column location, edge or corner, and cyclic behavior under different levels of slab gravity loads.

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List of Symbols

d	Slab effective depth
e	Eccentricity of the applied load
f_c'	150*300 mm concrete cylinder compressive strength
f_{ctd}	TS500 design tensile strength of concrete
M_{un}	Unbalanced moment from the structural analysis
P_{DEC2}	Slab punching capacity according to Draft Final EC-2
P_{EC2}	Slab punching capacity according to EC-2
P_{mod}	Modified punching capacity according TS-500
P_{TS500}	Slab punching capacity according to TS-500
r	Square column dimension
u_p	Critical punching perimeter
v_{ed}	Design shear stress
V_{sd}	Shear design force for EC-2
V_{un}	Unbalanced shear force from structural analysis
V_e	Experimental punching shear capacity
W	Critical perimeter property analogues to the polar moment of inertia
β	Reduction coefficient for eccentricity according to EC-2
γ	Coefficient for the eccentricity and column rectangularity in TS-500
ρ_l	Reinforcement ratio of tension steel
τ_{rd}	Design shear stress for EC-2

Table 1. Test Specimen Properties and Capacities

Data	Author	Specimens	$f_c^{'}$ (MPa)	r (mm)	d (mm)	ρ (%)	V_e (kN)	M_e (kNm)
1	Ertas	RCNR1E1	22.53	200	100	0.68	182.39	18.24
2	Ertas	RCNR1E2	22.91	200	100	0.68	118.11	23.62
3	Ertas	RCNR2E1	21.47	200	100	1.00	185.06	18.51
4	Ertas	RCNR2E2	20.21	200	100	1.00	137.46	27.49
5	Ozden	NR1E1F0	19.29	200	100	0.70	158.60	15.86
6	Ozden	NR1E2F0	18.46	200	100	0.70	117.62	23.52
7	Ozden	NR2E1F0	20.85	200	100	1.10	178.35	17.84
8	Ozden	NR2E2F0	20.12	200	100	1.10	130.06	26.01
9	Stamenkovic	C/I/1	38.3	127	55.6	1.20	84.59	7.33
10	Stamenkovic	C/I/2	31.6	127	55.6	1.20	62.32	10.49
11	Stamenkovic	C/I/3	27.2	127	55.6	1.20	33.83	13.65
12	Moe	M2A	15.5	304.8	114.3	1.50	212.8	40.54
13	Moe	M4A	17.7	304.8	114.3	1.50	143.8	62.09
14	Moe	M2	25.7	304.8	114.3	1.50	292.49	55.72
15	Moe	M3	22.7	304.8	114.3	1.50	207.45	68.50
16	Moe	M7	24	254	114.3	1.30	311.63	19.79
17	Moe	M8	24.6	254	114.3	1.30	149.58	64.59
18	Moe	M9	23.3	254	114.3	1.30	267.12	33.92
19	Moe	M10	21.1	254	114.3	1.30	178.08	54.28
20	Moe	H6	26.5	254	114.3	1.30	239.51	39.54
21	Marzouk	NNHS 1.0	36.2	250	119	1.00	163.6	117.47
22	Marzouk	NHHS 1.0	35.3	250	119	1.00	250.3	116.14
23	Hanson	A12	33.3	152.4	57.2	1.50	26.89	20.52
24	Hanson	A13L	32.82	152.4	57.2	1.50	26.15	19.89
25	Durrani	DYN-2	25.72	254	96.77	0.59	85.41	33.44
26	Durrani	DYN-4	19.11	254	96.77	0.59	52.93	44.06
27	Lou	INT 1	30.96	254	96.77	0.55	139.67	39.21
28	Lou	INT 2	30.68	254	96.77	0.55	152.13	31.64
29	Robertson	B	30.75	254	91.44	0.83	85.85	41.35
30	Robertson	C	32.2	254	91.44	0.83	120.55	27.12
31	Zee	INT	26.2	137.16	51.56	0.65	16.01	10.28
32	Pan	1	33.3	274.32	103.8	0.72	111.21	74.80
33	Pan	3	31.37	274.32	103.8	0.72	48.93	105.53
34	Hawkins	S1	34.82	304.8	114.3	1.18	128.11	144.62
35	Hawkins	S2	23.44	304.8	117.6	0.79	142.34	97.90
36	Hawkins	S3	22.06	304.8	120.65	0.51	138.78	53.67
37	Hawkins	S4	32.34	304.8	114.3	1.18	149.91	125.41
38	Symonds	S6	23.17	304.8	114.3	1.71	267.78	72.76
39	Symonds	S7	26.48	304.8	117.6	0.83	72.76	42.48
40	Ghali	SM 0.5	36.8	305	120.7	0.50	129	99.98
41	Ghali	SM 1.0	33.4	305	120.7	1.00	129	127.97
42	Ghali	SM 1.5	39.9	305	120.7	1.50	129	133.00
43	Emam	NHCC 0.5	36.76	250	119	0.50	125	100.48
44	Emam	NHCC 1.0	35.37	250	119	1.00	125	127.24
45	Elgabry	1	35	250	116	1.10	150	57.00
46	Wey	SC-0	20.7	254	96.77	1.00	66.29	62.08

Table 2. Capacity Predictions according to Codes.

Data	e/r	P_{TS500} (kN)	P_{mod} (kN)	P_{EC2}(kN)	P_{DEC2}(kN)	V_e/P_{TS500}	V_e/P_{mod}	V_e/P_{EC2}	V_e/P_{DEC2}
1	0.500	166.88	166.88	140.11	141.01	1.093	1.093	1.302	1.293
2	1.000	144.71	144.71	141.68	114.99	0.816	0.816	0.834	1.027
3	0.500	162.91	162.91	147.48	157.80	1.136	1.136	1.255	1.173
4	1.000	135.92	135.92	141.65	125.41	1.011	1.011	0.970	1.096
5	0.500	154.42	154.42	127.02	135.20	1.027	1.027	1.249	1.173
6	1.000	129.90	129.90	123.35	108.04	0.905	0.905	0.954	1.089
7	0.500	160.54	160.54	148.24	161.31	1.111	1.111	1.203	1.106
8	1.000	135.61	135.61	144.76	129.27	0.959	0.959	0.898	1.006
9	0.682	68.82	68.82	77.23	59.53	1.229	1.229	1.095	1.421
10	1.325	51.87	38.90	67.93	43.14	1.202	1.602	0.917	1.444
11	3.177	32.29	24.22	61.47	24.80	1.048	1.397	0.550	1.364
12	0.625	208.27	208.27	199.22	213.68	1.022	1.022	1.068	0.996
13	1.417	175.59	131.69	217.65	159.66	0.819	1.092	0.661	0.901
14	0.625	268.18	268.18	279.08	252.91	1.091	1.091	1.048	1.157
15	1.083	218.24	218.24	256.91	197.14	0.951	0.951	0.807	1.052
16	0.250	262.23	262.23	232.24	273.09	1.188	1.188	1.342	1.141
17	1.700	173.28	129.96	236.09	151.79	0.863	1.151	0.634	0.985
18	0.500	236.67	236.67	227.70	237.13	1.129	1.129	1.173	1.126
19	1.200	182.32	182.32	213.13	170.62	0.977	0.977	0.836	1.044
20	0.650	240.28	240.28	248.10	230.50	0.997	0.997	0.965	1.039
21	2.872	173.01	129.76	298.87	126.03	0.946	1.261	0.547	1.298
22	1.856	210.48	157.86	293.89	161.82	1.189	1.586	0.852	1.547
23	5.007	30.81	23.10	86.22	21.54	0.873	1.164	0.312	1.249
24	4.991	30.65	22.99	85.39	21.49	0.853	1.138	0.306	1.217
25	1.541	145.43	109.07	160.21	90.72	0.587	0.783	0.533	0.941
26	3.277	86.66	64.99	131.42	51.51	0.611	0.814	0.403	1.028
27	1.105	179.73	179.73	179.27	110.86	0.777	0.777	0.779	1.260
28	0.819	195.11	195.11	178.18	124.97	0.780	0.780	0.854	1.217
29	1.896	134.58	100.93	177.81	86.33	0.638	0.851	0.483	0.994
30	0.886	181.31	181.31	183.36	127.74	0.665	0.665	0.657	0.944
31	4.681	23.21	17.41	48.56	12.88	0.690	0.920	0.330	1.243
32	2.452	154.86	116.15	226.65	92.67	0.718	0.958	0.491	1.200
33	7.862	70.60	52.95	217.80	37.60	0.693	0.924	0.225	1.301
34	3.704	152.99	114.74	317.41	101.27	0.837	1.116	0.404	1.265
35	2.257	171.92	128.94	230.01	114.89	0.828	1.104	0.619	1.239
36	1.269	219.94	164.95	212.03	140.02	0.631	0.841	0.655	0.991
37	2.745	175.28	131.46	302.15	121.89	0.855	1.140	0.496	1.230
38	0.891	233.60	233.60	272.60	225.02	1.146	1.146	0.982	1.190
39	1.915	197.33	148.00	252.12	134.21	0.369	0.492	0.289	0.542
40	2.541	210.72	158.04	297.66	111.91	0.612	0.816	0.433	1.153
41	3.253	175.37	131.52	318.89	115.68	0.736	0.981	0.405	1.115
42	3.380	187.41	140.56	403.90	136.75	0.688	0.918	0.319	0.943
43	3.215	163.91	122.93	264.20	93.36	0.763	1.017	0.473	1.339
44	4.072	139.91	104.93	294.28	98.56	0.893	1.191	0.425	1.268
45	1.520	218.73	164.04	288.67	175.70	0.686	0.914	0.520	0.854
46	3.687	84.06	63.05	154.45	57.96	0.789	1.051	0.429	1.144

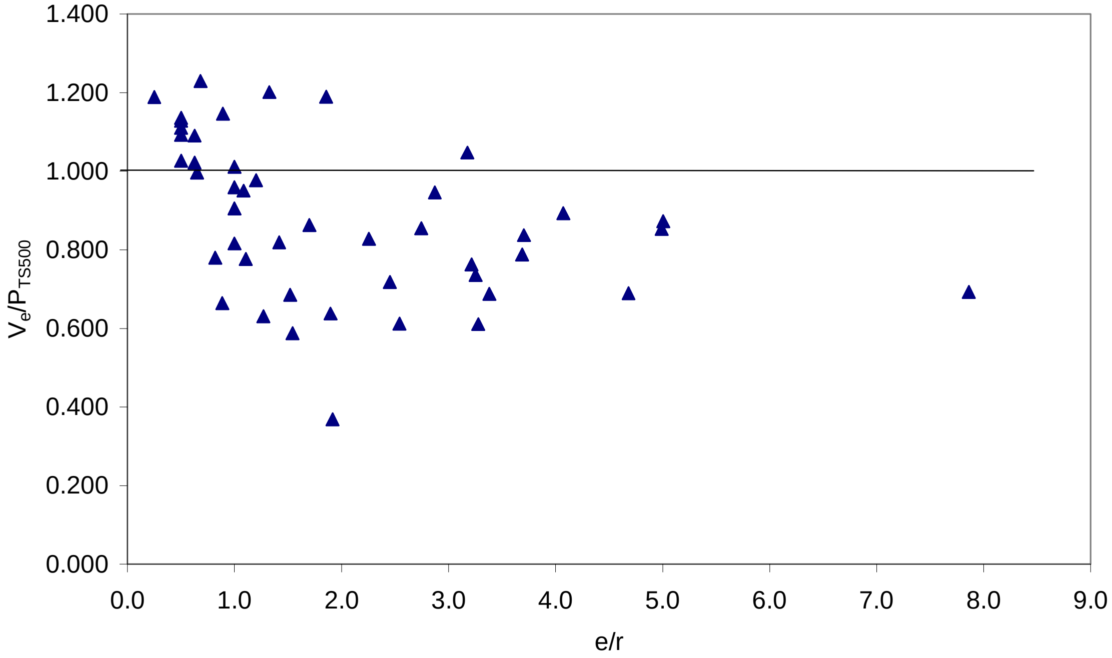

Figure 1. The variation of predictions with e/r according to TS-500

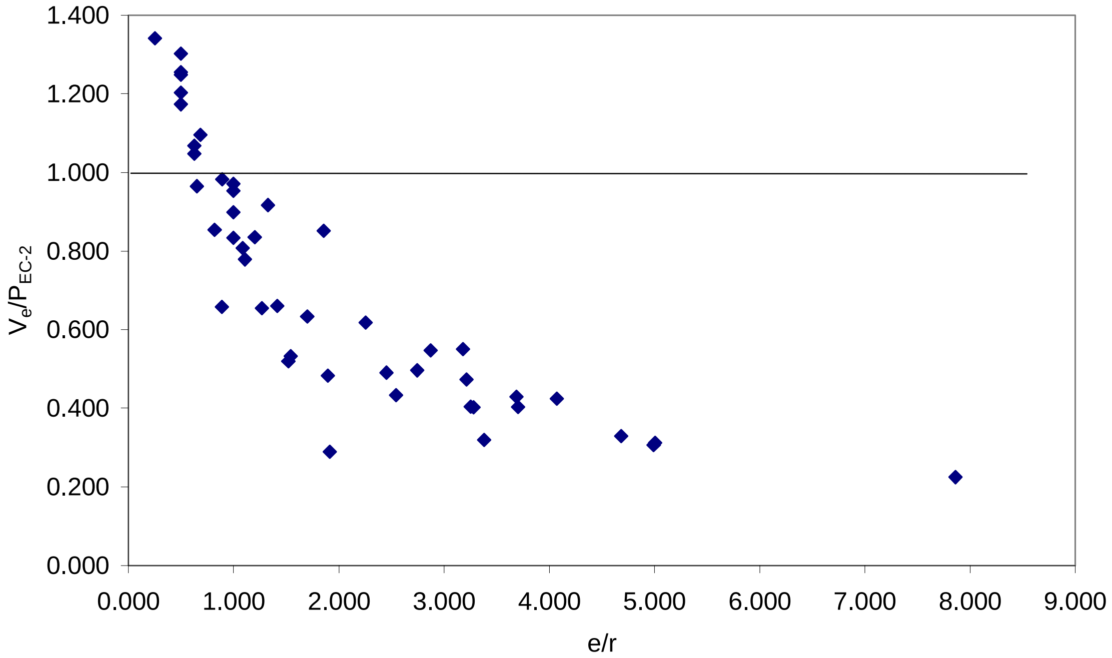

Figure 2. The variation of predictions with e/r according to EC-2

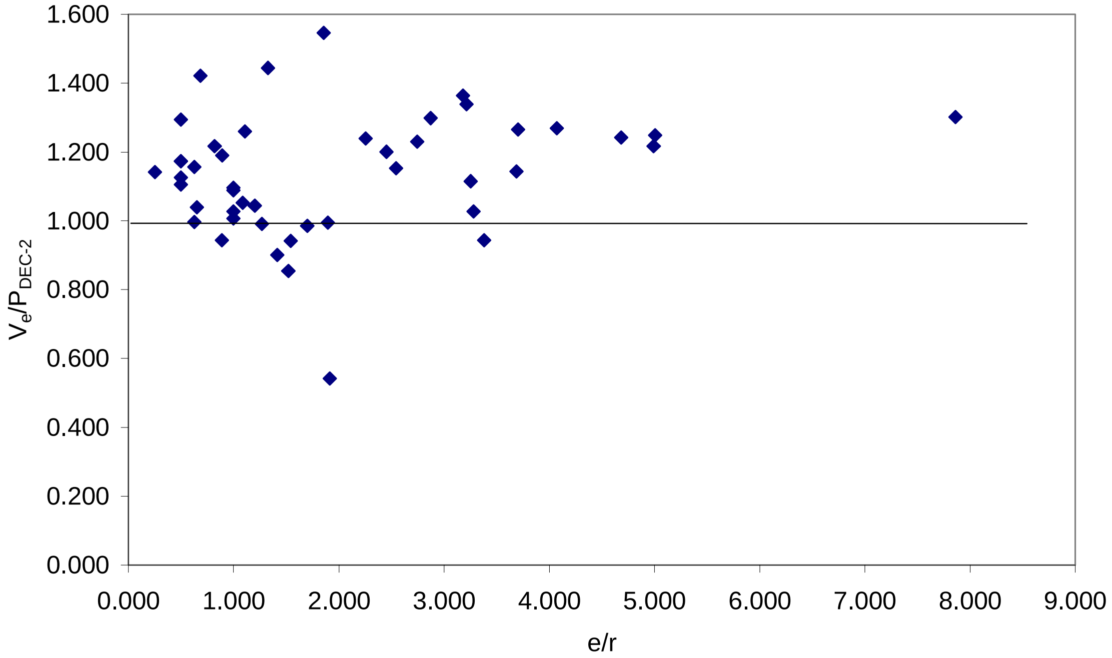

Figure 3. The variation of predictions with e/r according to Final Draft EC-2

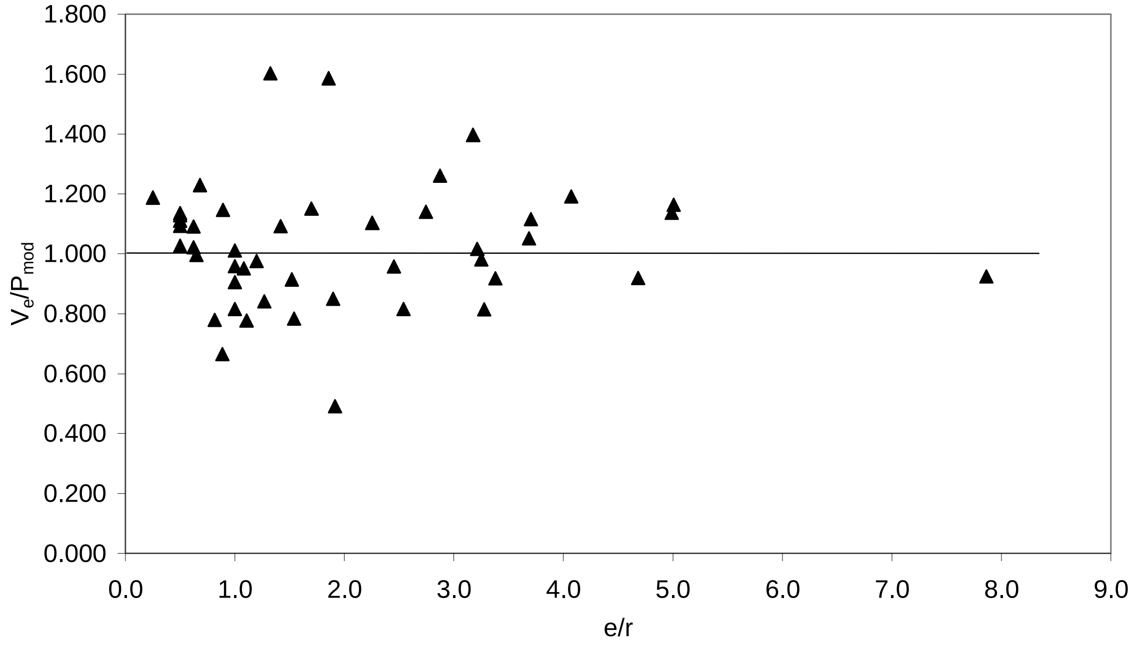

Figure 4. Modification of TS-500