TABLE 2.1

List of Peak Ground Acceleration Models
(Names of models refer to the Classification in
Questionnaire Q4 - or Interim Report, Appendix C, Section 7),
and the regions are shown on the map of Figure 2.1.

Class of Model	Model Name	Index in file of Appendix A	Distinguish Rock - Soil	Expert for whom it is a best estimate model (per region) Region NE	SE	NC	SC
1 (Intensity-no weighting)	A1-G16	8	No				
	A3-G15	1	No				
	A3-G16	12	Yes				
	A4-G12	4	No				
	A4-G16	16	Yes				
	A5-G16	20	Yes				
	A6-G16	27	Yes	5	5	5	5
2 (Intensity - Distance Weighting)	A1-G21	9	No				
	A1-G22	30	Yes				
	A3-G21	13	No				
	A3-G22	32	Yes				
	A4-G21	17	No				
	A4-G22	31	Yes				
	A5-G21	21	No				
	A5-G22	29	Yes				
3 (Intensity-Magnitude Weighting)	A1-G31	10	No				
	A3-G31	3	No				
	A4-G31	18	No				
	A5-G31	22	No				

TABLE 2.1 (Continued)

Class of Model	Model Name	Index in file of Appendix A	Distinguish Rock - Soil	Expert for whom it is a best estimate model (per region) Region NE	SE	NC	SC
4 (Intensity - Magnitude & Dist. Weighting)	A1-G41	11	No				
	A3-G41	15	No				
	A4-G41	19	No				
	A5-G41	23	No				
5 (Semi-emperical)	G51	28	No			2	
	G52	24	No		2		2
	G53	5	No	2			
6 (Direct-1)	D12	6	No				
	D13	25	No	3	3	3	3
	D14	26	No				
7 (Direct-2)	D21	7	No	1,4	1,4	1,4	1,4

TABLE 2.2

List of Peak Ground Velocity Models
(Names of models refer to the Classification in Questionnaire Q4 - or Interim Report, Appendix C, Section 7), and the regions are shown on the map of Figure 2.1)

Class of Model	Model Name	Index in file of Appendix A	Distinguish Rock - Soil	Expert for whom it is a best estimate model (per region) Region NE	SE	NC	SC
1 (Intensity No Weighting)	A1-GV12	38	Yes				
	A3-GV12	33	Yes				
	A4-GV12	44	Yes				
	A5-GV12	47	Yes				
2 (Intensity Dist. Weighting)	A1-GV22	39	Yes				
	A3=GV21	34	No				
	A3-GV22	42	Yes				
	A4-GV22	45	Yes				
	A5-GV22	48	Yes				
3 (Intensity Mag. Weighting.	A1-GV31	40	No				
	A3-GV31	35	No				
	A4-GV31	46	No				
	A5-GV31	49	No				
4 (Intensity-Mag & Dist. weighting)	No models						
5 (Semi-empirical)	GV51	50	No			2	
	GV52	36	No		2		2
	GV53	52	No	2			
6 (Direct-1)	DV12	51	No				
7 (Direct-2)	DV21	37	No	1,3,4	1,3,4	1,3,4	1,3,4

TABLE 2.3

List of Pseudo Velocity Spectra Models
(Names of models refer to the classification in Questionnaire Q4, and Interim Report, Appendix C, Section 7.
The regions are shown on the map of Figure 2.1

Class of Model	Anchor equation Acceleration and/or Velocity	Index in file of Appendix A	Distinguish Rock - Soil	Expert for whom it is a best estimate model (per region) Region NE	SE	NC	SC
RS1 (RG 1.60)	G51	161	No				
	G52	170	No		2		2
	G53	179	No				
	D13	80	No				
	D21	71	No				
RS2 (NBS, 1978 ATC	G51	134	No			2	
	G52	143	No				
	G53	152	No	2			
	D13	98	No				
	D21	89	No				
RS3 (Newmark Hall)	G51/GV51	188	No				
	G52/GV52	197	No				
	G53/GV53	206	No				
	D13/DV21	116	No	3	3	3	3
	D221/DV21	107	No	1,4	1,4	1,4	1,4
RS4 (Dist. Weight)	SEP 1 Bernreuter	53	No				
RS5 (Mag-weighted)	SEP 2 Bernreuter	62	No				
RS6 (No weighting)	Trifunac-Anderson	125	Yes	5	5	5	5

TABLE 2.4

Random Uncertainty Input
Values of "sigma" used in the analyses[1]

Expert Index		Acceleration Region				Velocity Region				Spectra Region			
		NE	SE	NC	SC	NE	SE	NC	SC	NE	SE	NC	SC
1	Low	.35				.40				.45			
	Best	.5				.55				.60			
	Up	.65				.70				.75			
2	Low	.4				.3				.4			
	Best	.6				.5				.6			
	Up	.8				.7				.8			
3	Low	.48				.64				.53			
	Best	.6				.76				.65			
	Up	.72				.88				.77			
4	Low	.5	.5	.5	.5	.5	.5	.5	.5	.7			
	Best	.6	.6	.55	.6	.6	.6	.55	.6	.9			
	Up	.8	.8	.7	.8	.8	.8	.7	.8	1.4			
5[2]	Low	.6								.6			
	Best	.6								.6			
	Up	.6								.6			

Notes: 1. Values of signma not given for SE, NC, and SC are identical to those of NE.
2. Expert 5 did not provide values for velocity.

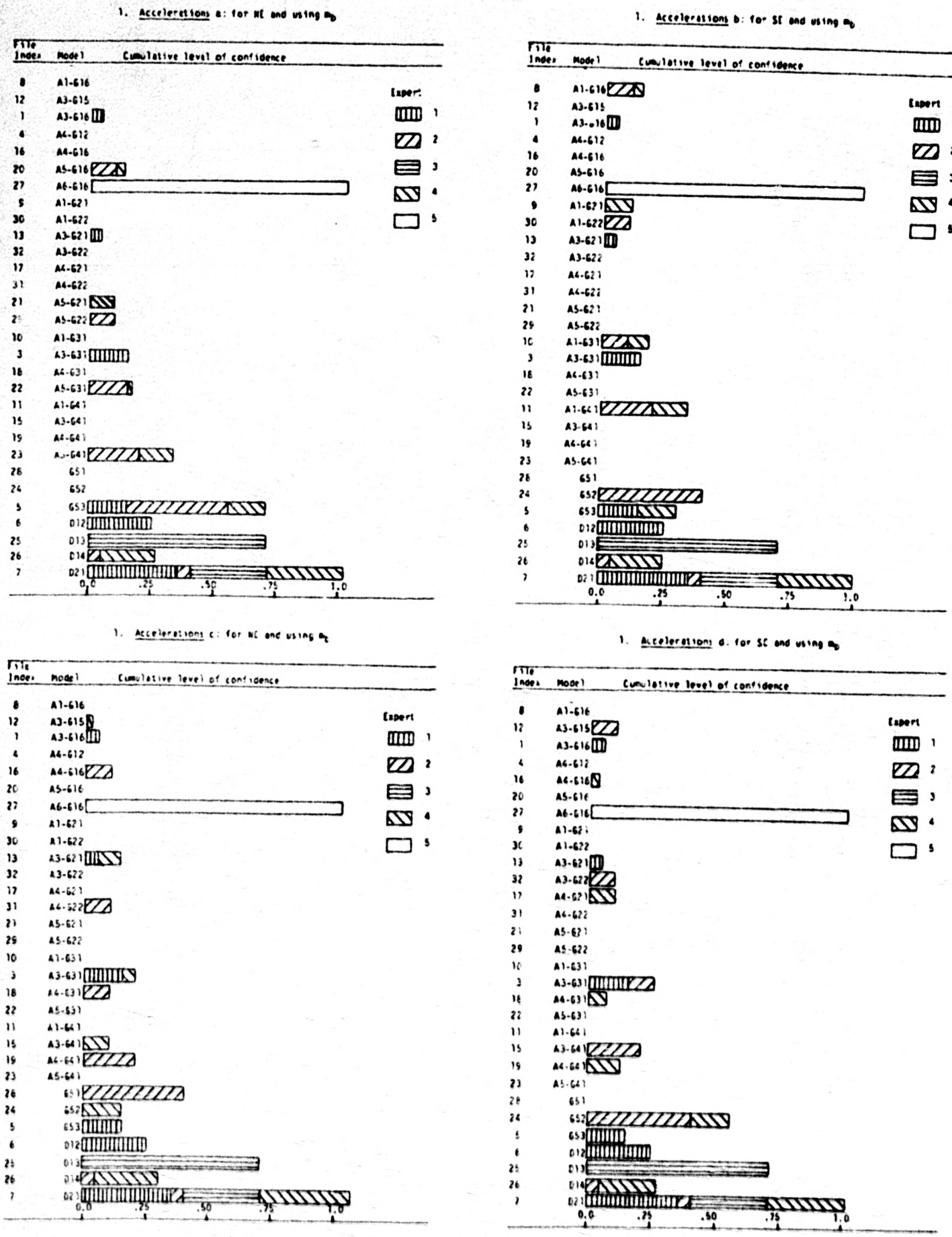
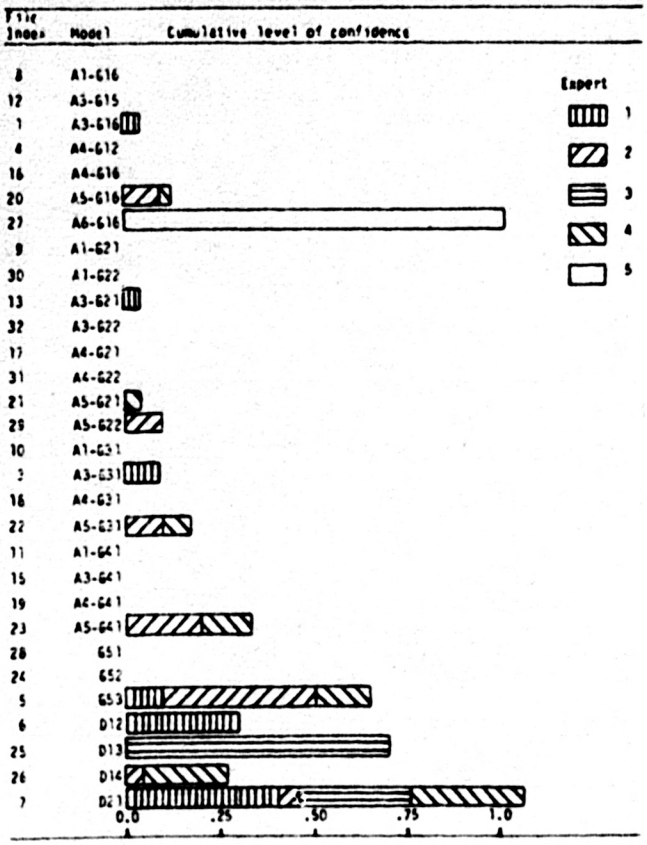
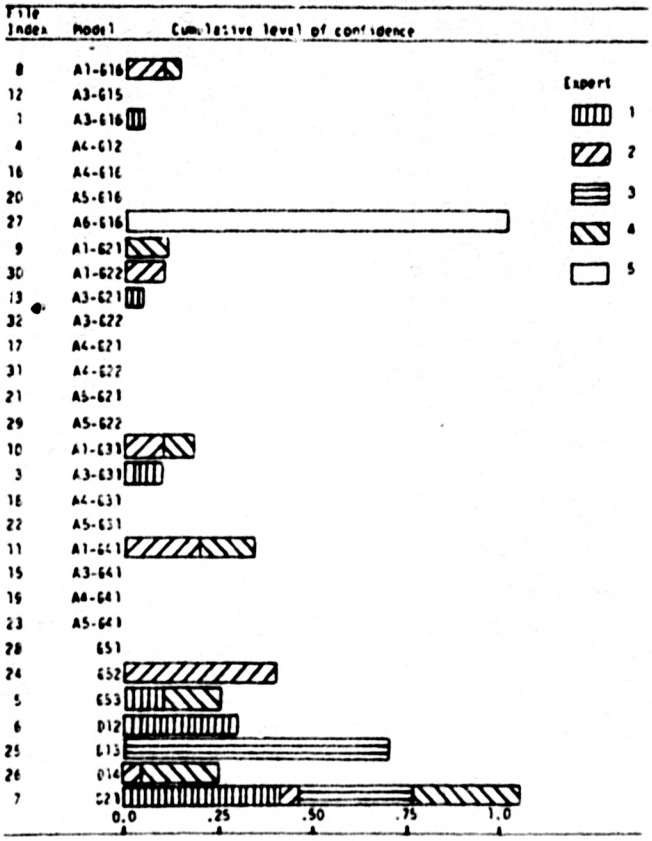


Table 2.5 Cumulative values of the levels of confidence assigned to the models in the analysis, for acceleration and spectra.

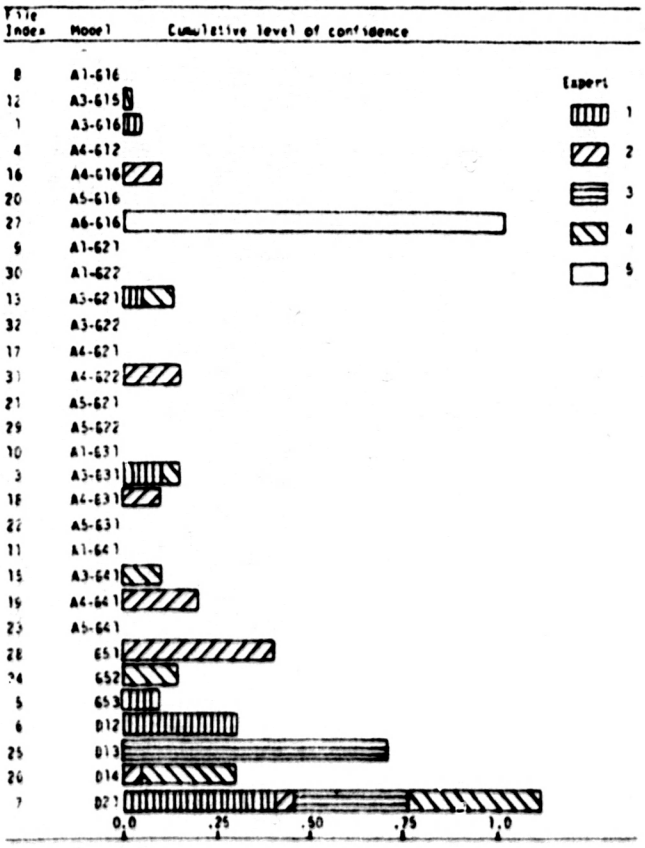
1. Accelerations e: for NF and using MMI



1. Accelerations f: for SF and using MMI



1. Accelerations g: for NC and using MMI



1. Accelerations h: for SC and using MMI

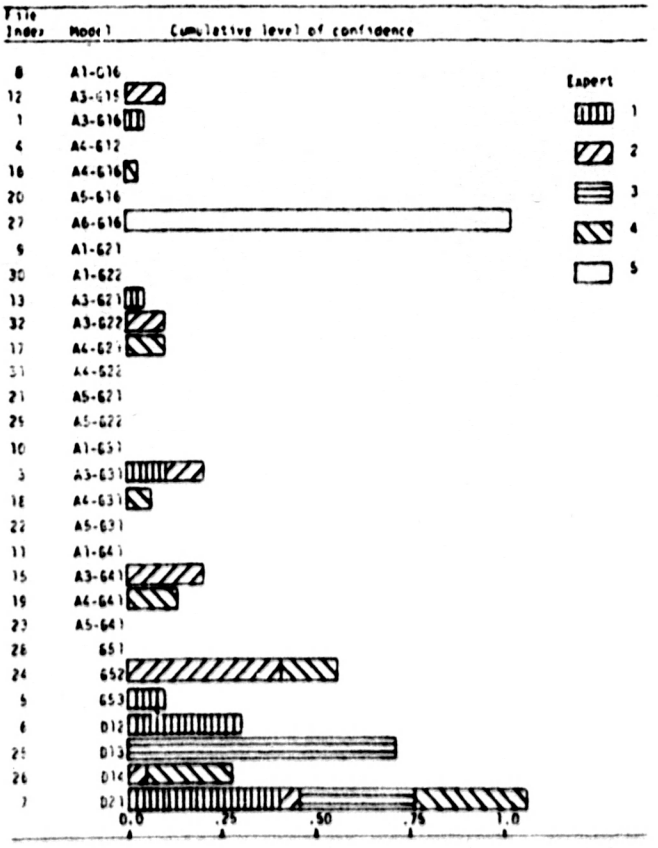
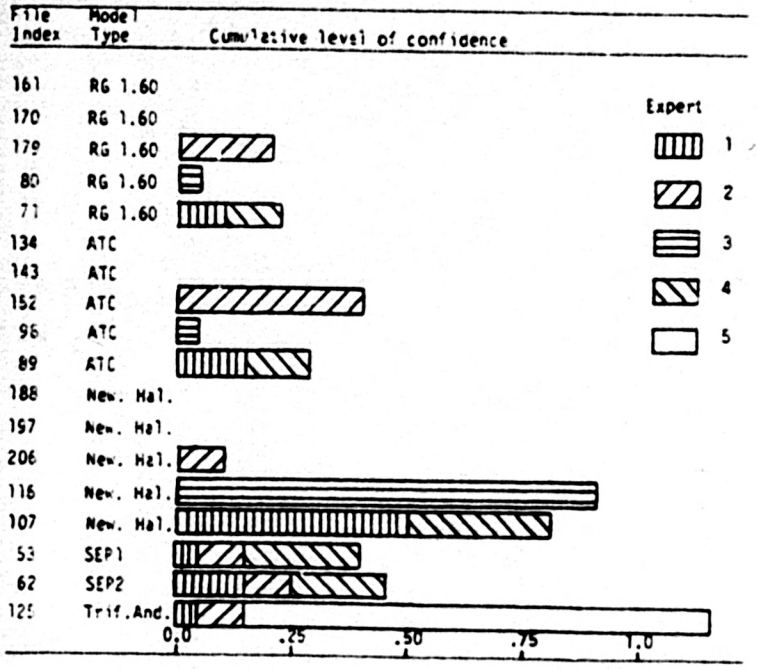
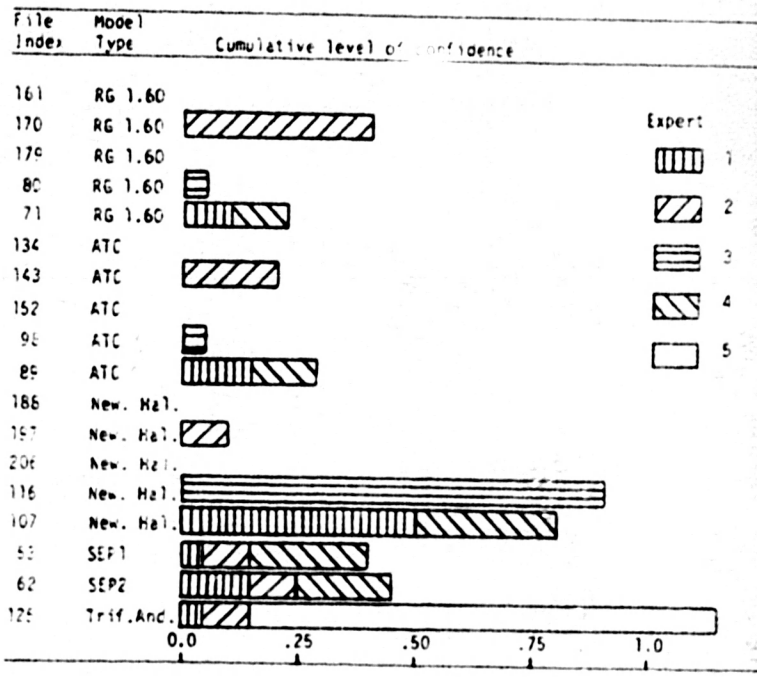


Table 2.5 (continued)

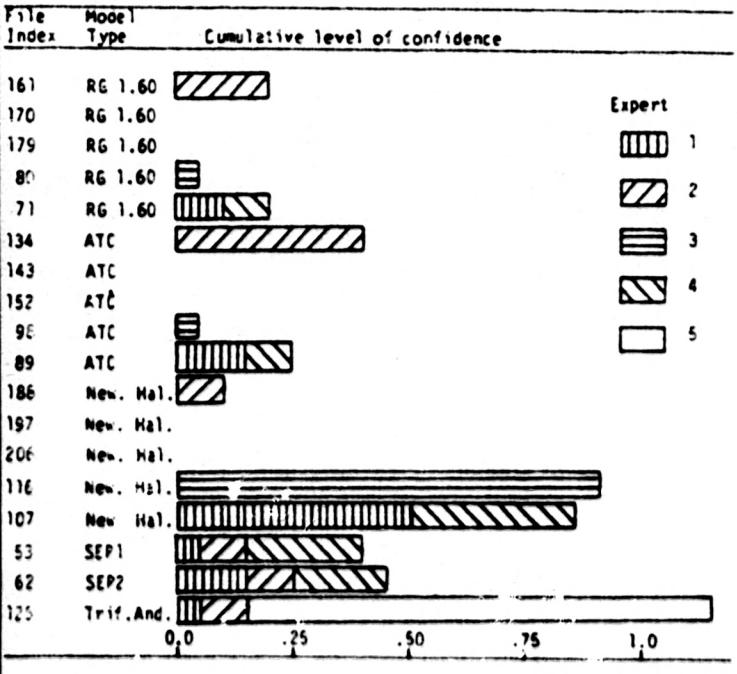
2. Spectra a: for NE and using m_b



2. Spectra b: for SE and using m_b



2. Spectra c: for NC and using m_b



2. Spectra d: for SC and using m_b

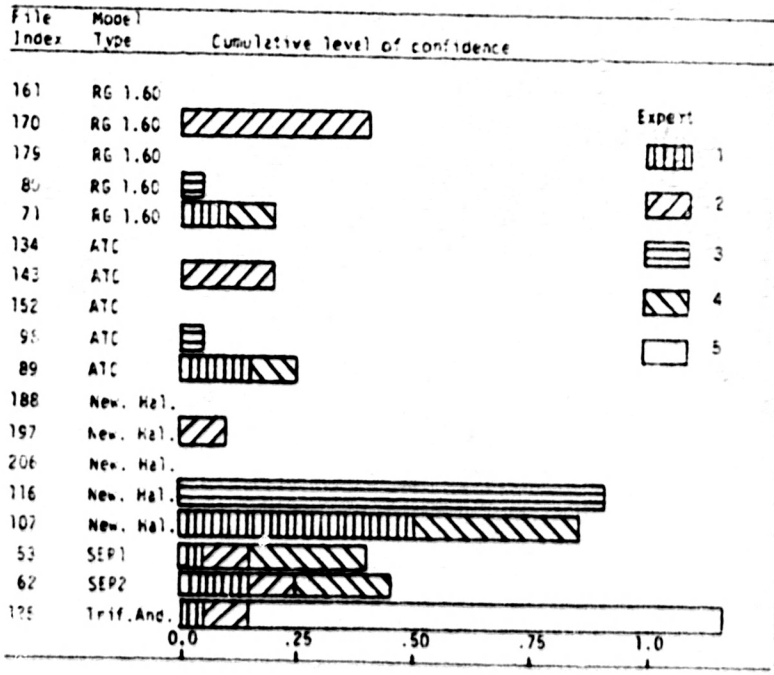
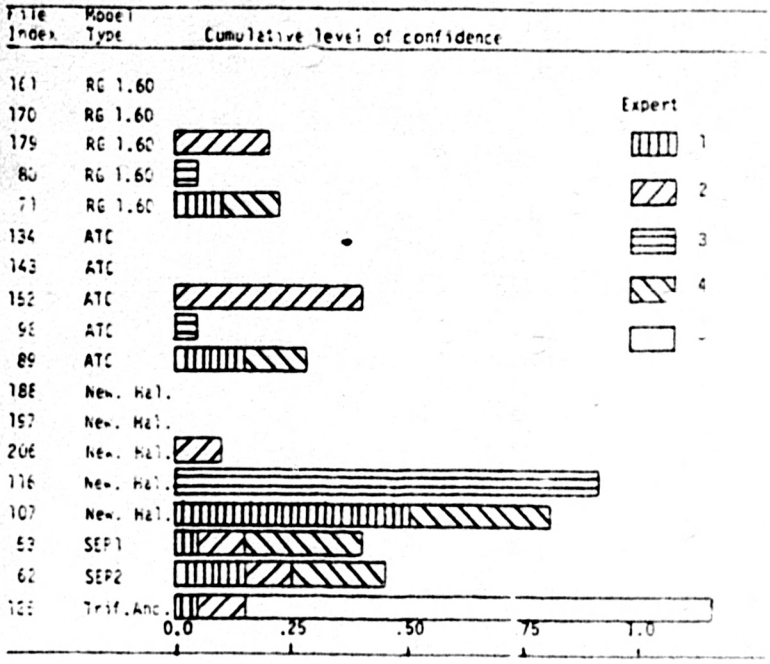
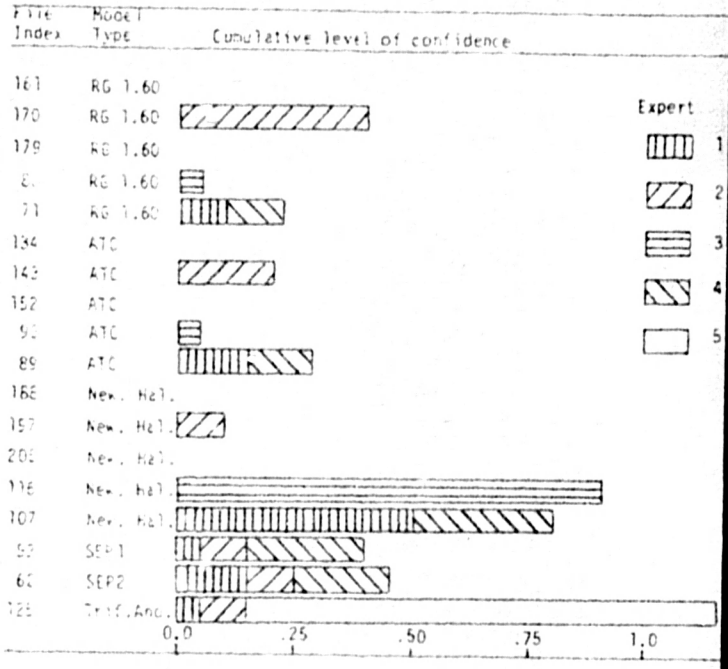


Table 2.5 (continued)

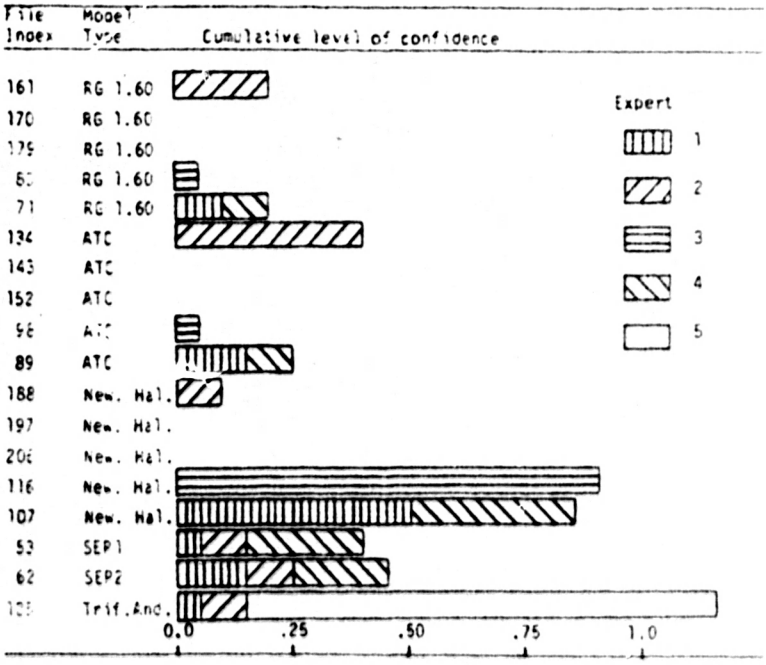
2. Spectra e: for NE and using MMI



2. Spectra f: for SE and using MMI



2. Spectra g: for NC and using MMI



2. Spectra h: for SC and using MMI

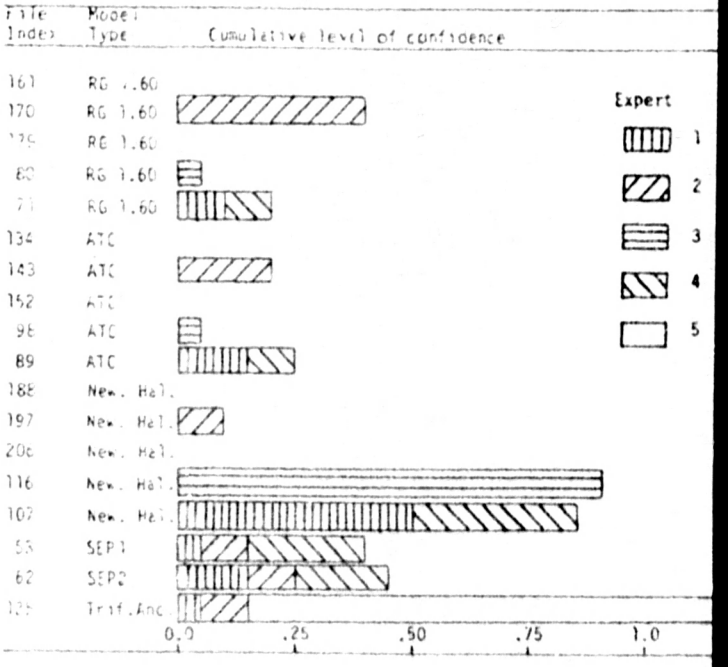


Table 2.5 (continued)

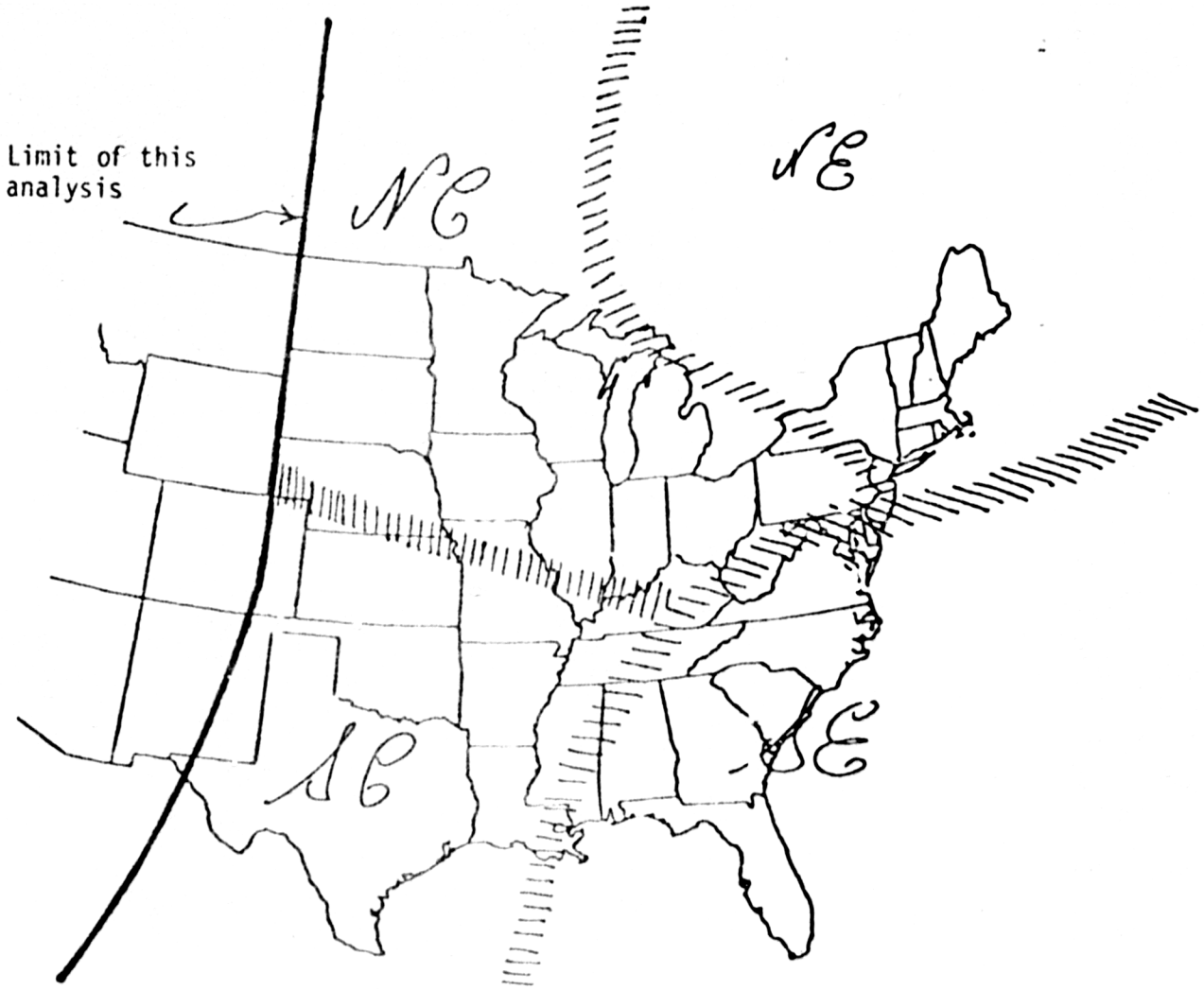


Fig. 2.1 Identification of four regions of the Eastern U.S. based on a compilation of the seismic zonation expert maps developed in this study, combined with a map of Q_o-contours from Singh & Herrmann (1983).

Plot Symbol	Model Number
1	1
2	4
3	8
4	12
5	16
6	20
7	27

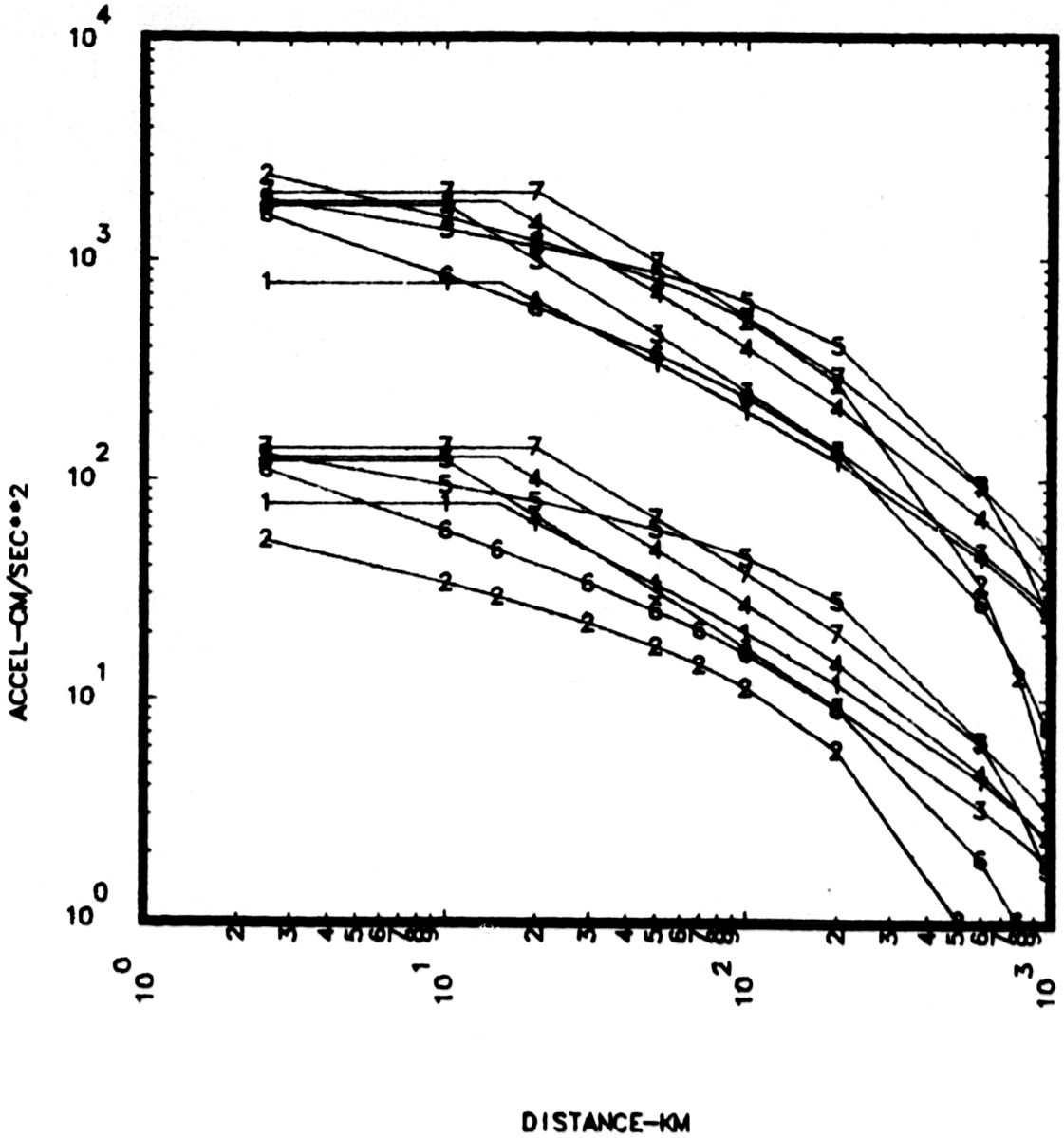


Fig. 2.2 Class 1 Intensity Based - No Weighting Acceleration Models Selected By GMP.

Plot Symbol	Model Number
1	9
2	13
3	17
4	21
5	29
6	30
7	31
8	32

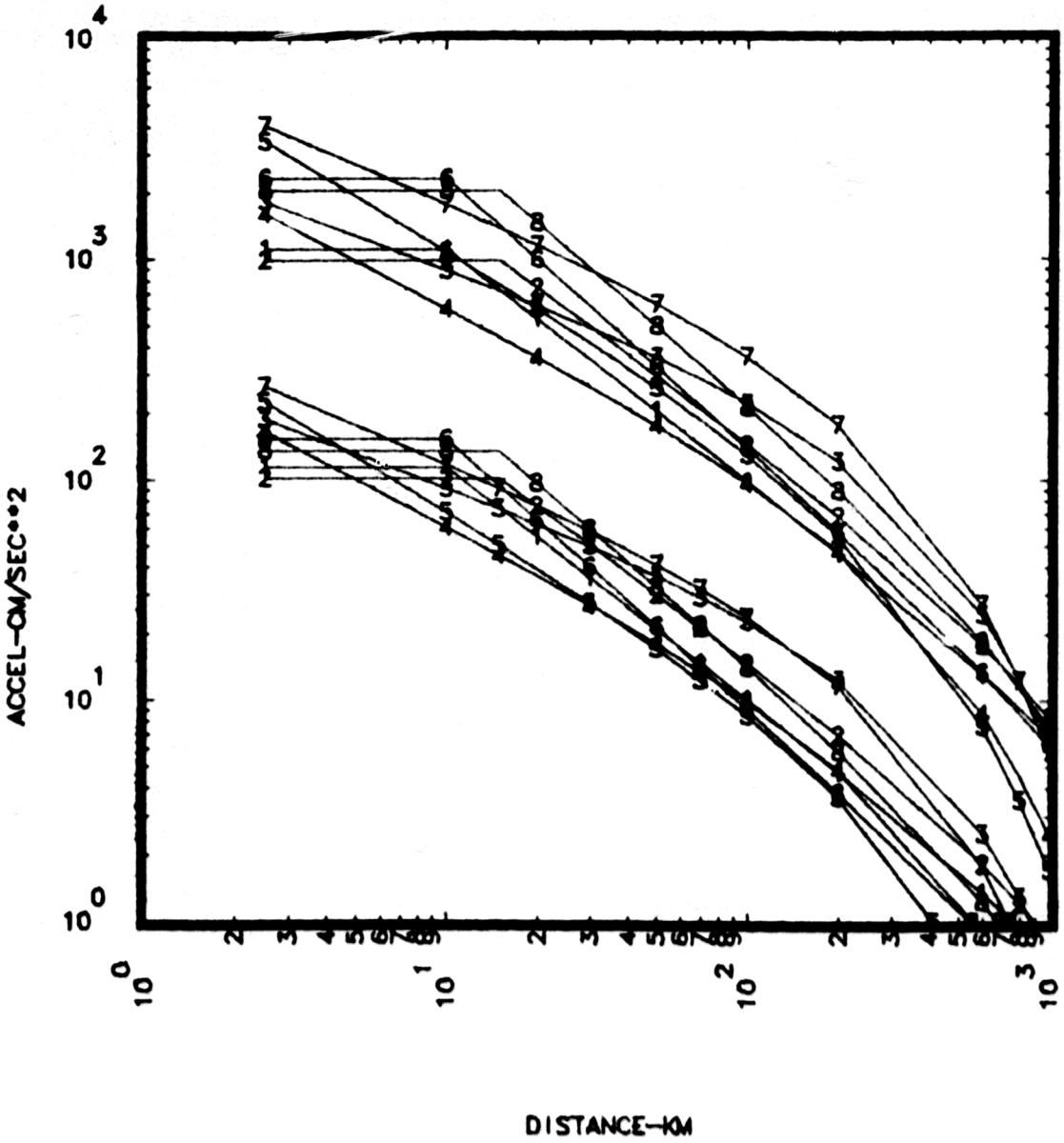


Fig. 2.3 Class 2 Intensity Based - Distance Weighting Acceleration Models Selected By GMP.

Plot Symbol	Model Number
1	3
2	10
3	18
4	22

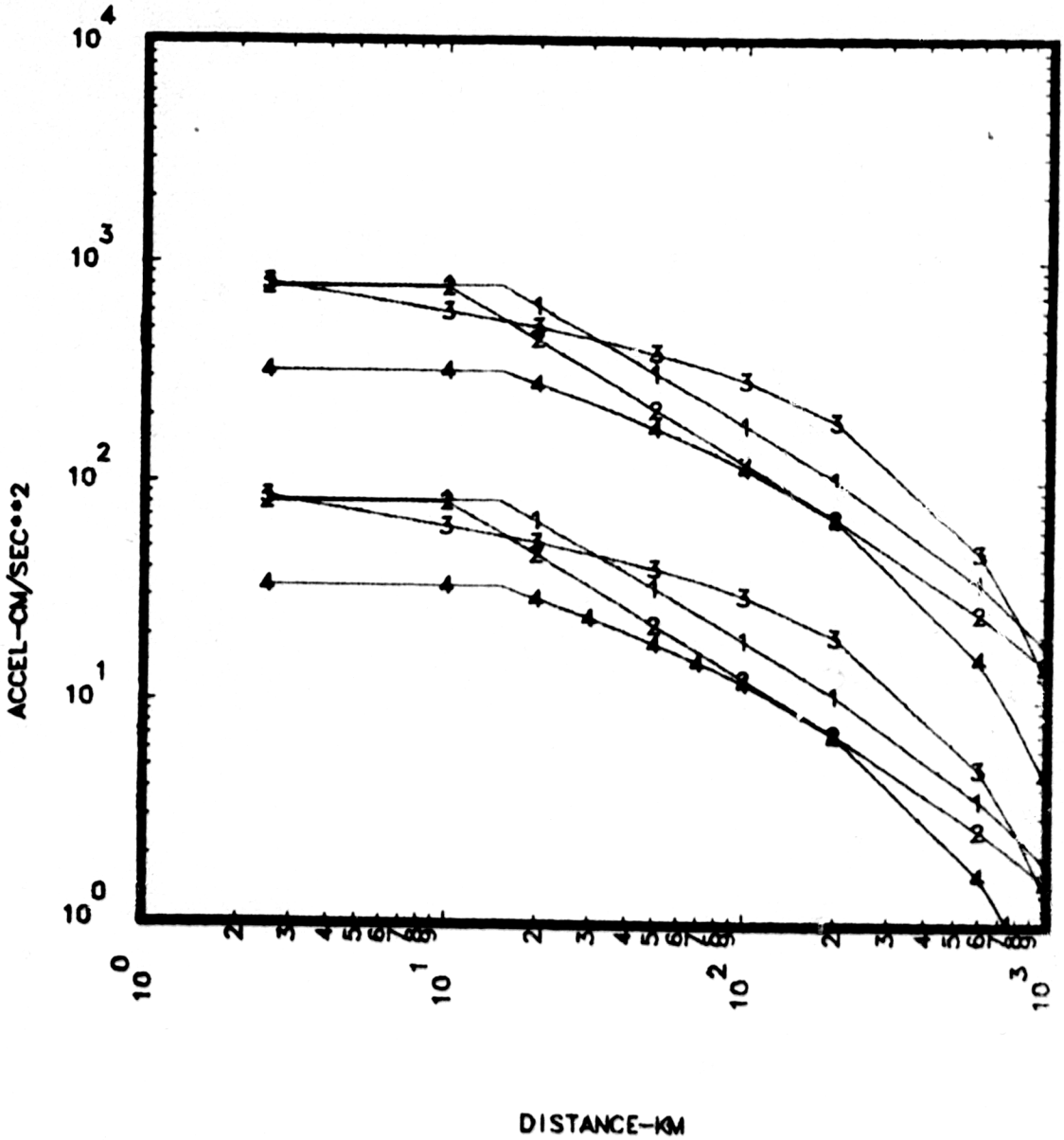


Fig. 2.4 Class 3 Intensity Based - Magnitude Weighting Acceleration Models Selected By GMP.

Plot Symbol	Model Number
1	11
2	15
3	19
4	23

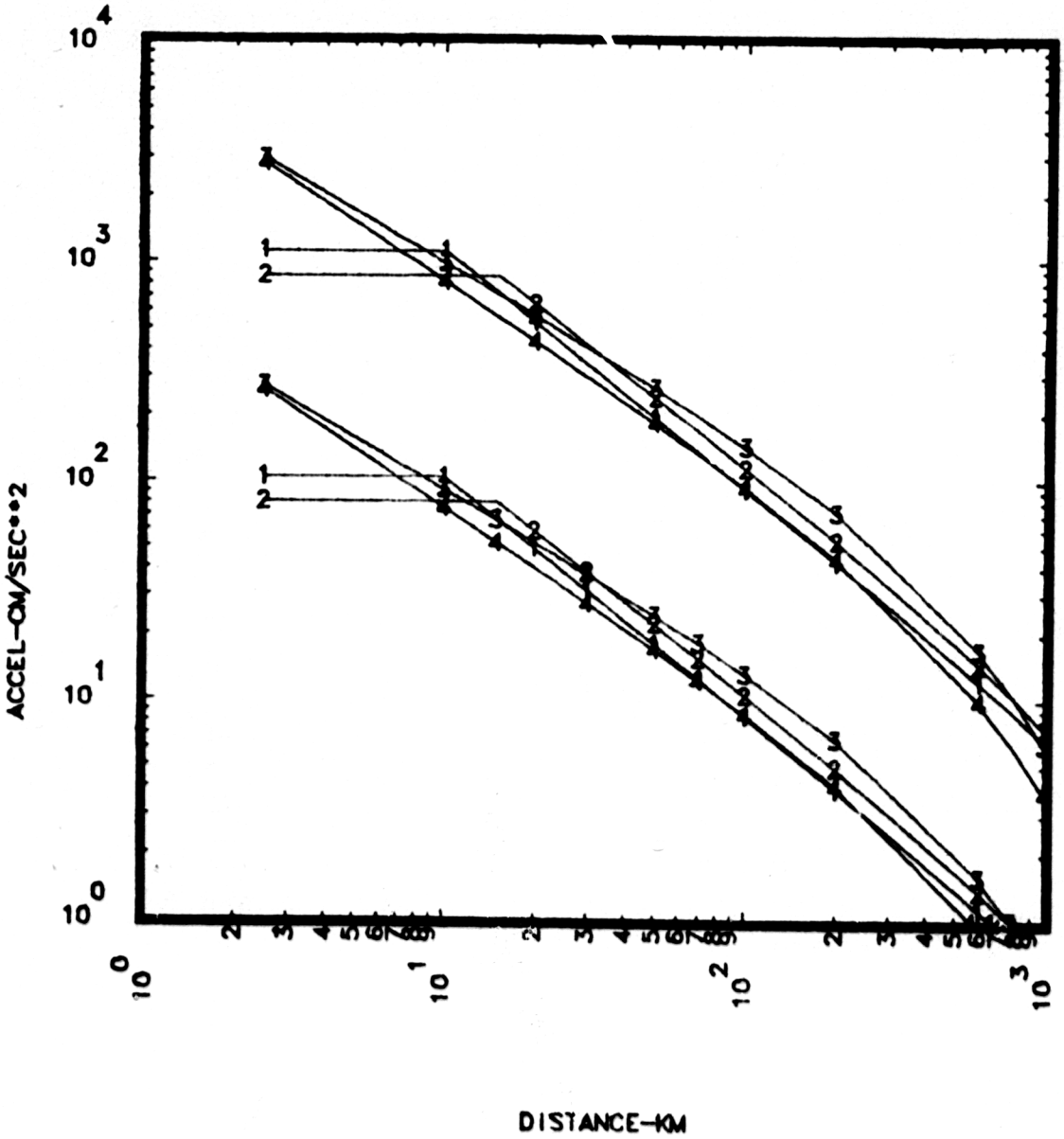


Fig. 2.5 Class 4 Intensity Based - Both Magnitude And Distance Weighting Acceleration Models Selected By GMP.

Plot Symbol	Model Number
1	5
2	24
3	28

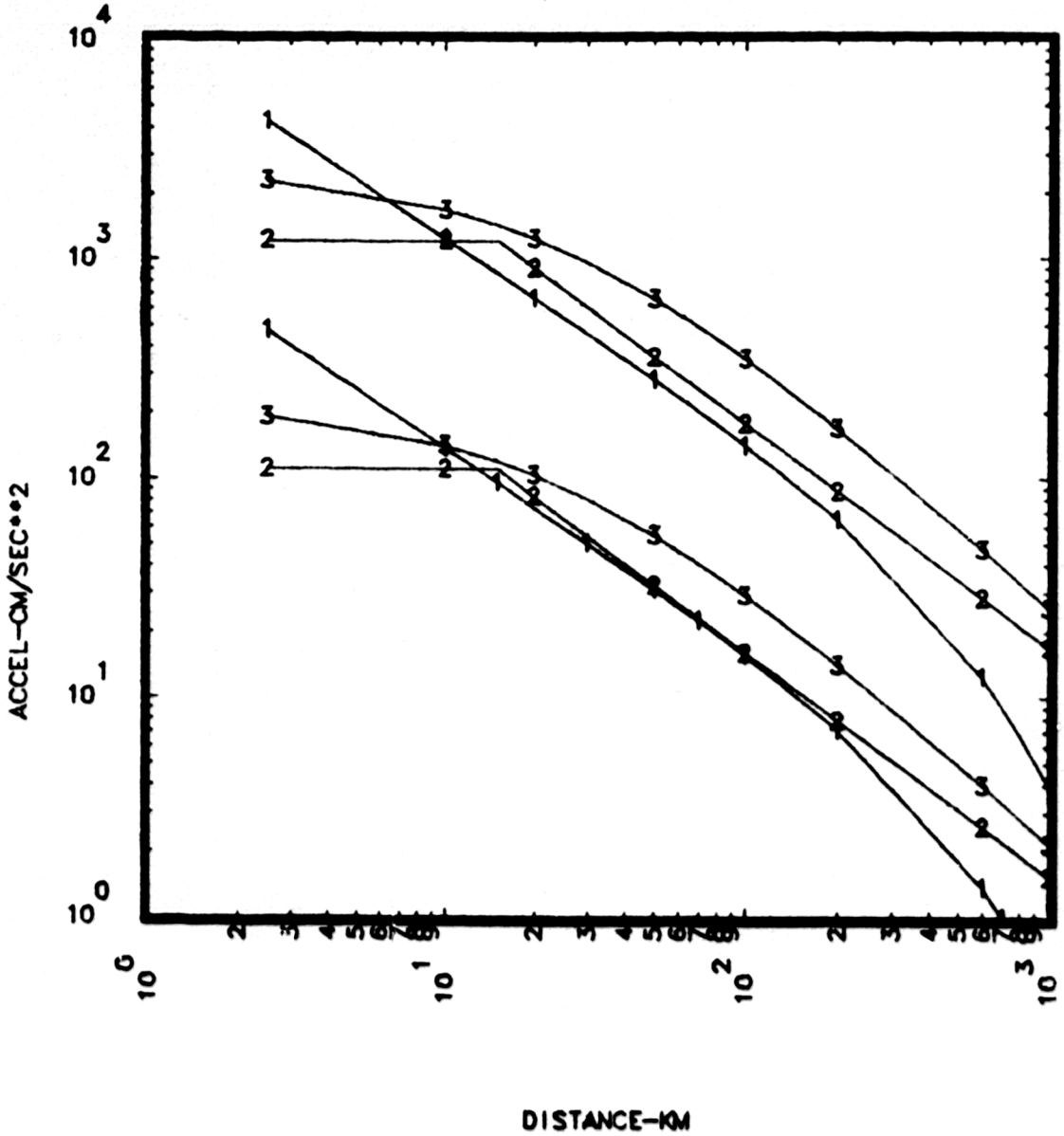


Fig. 2.6 Class 5 Intensity Based - Semi-Empirical Acceleration Models Selected By GMP.

Plot Symbol	Model Number
1	6
2	25
3	26
4	7

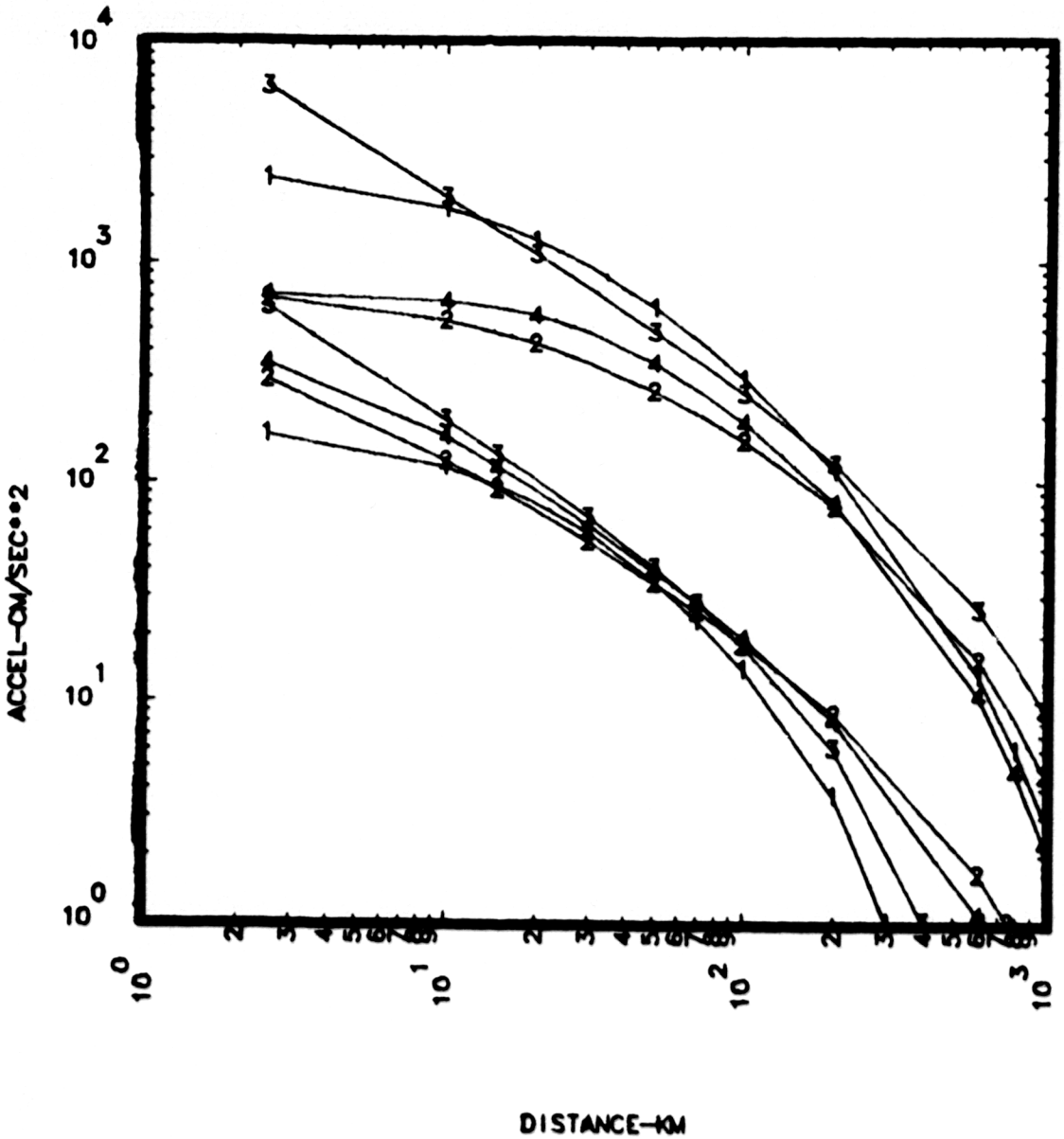


Fig. 2.7 Classes 6 & 7 - Direct Acceleration Models Selected By GMP.

Plot Symbol	Model Number
2	38
3	33
4	44
5	47

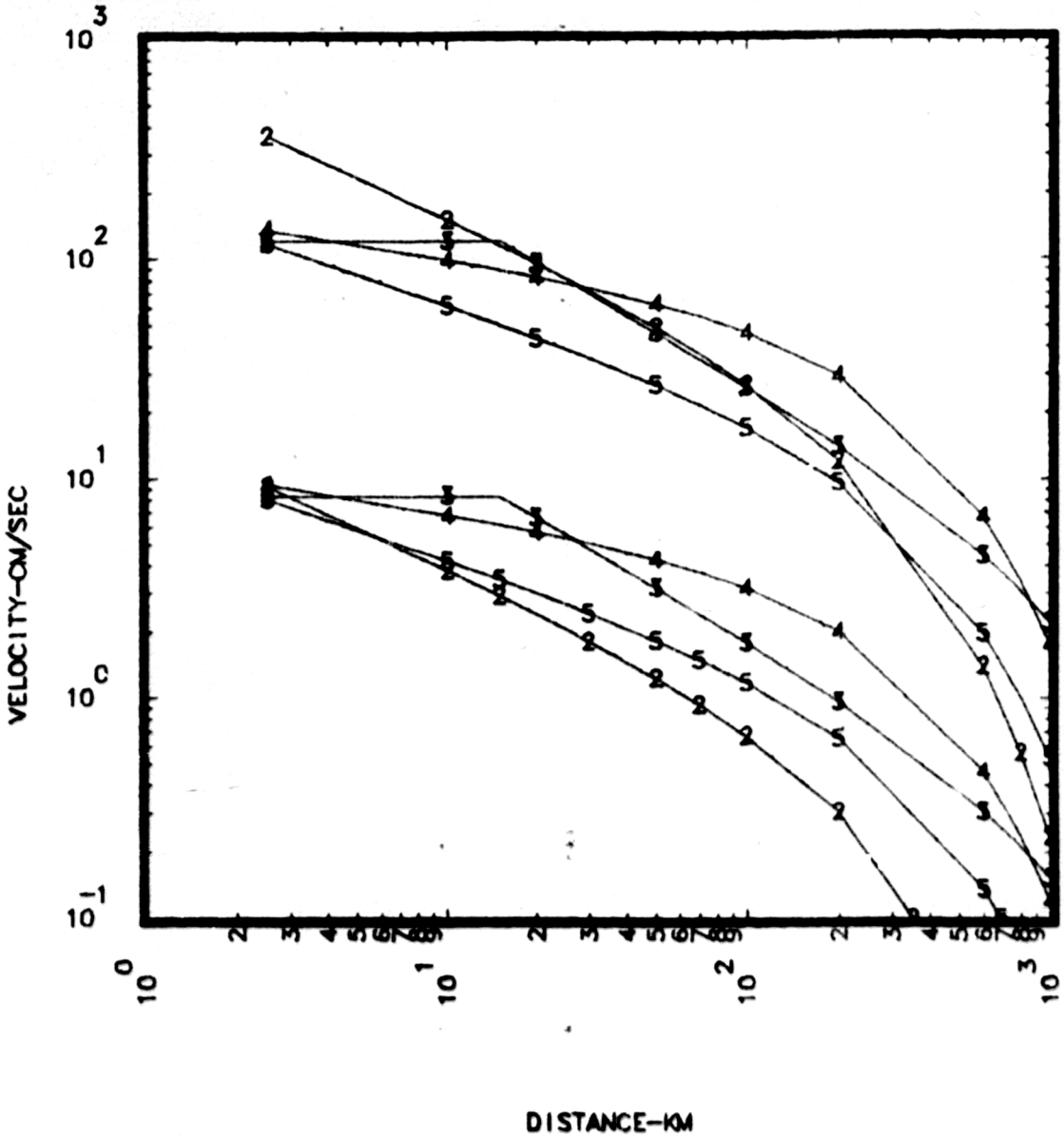


Fig. 2.8 Class 1 Intensity Based - No Weighting Velocity Models Selected by GMP.

Plot Symbol	Model Number
1	34
2	39
3	42
4	45
5	48

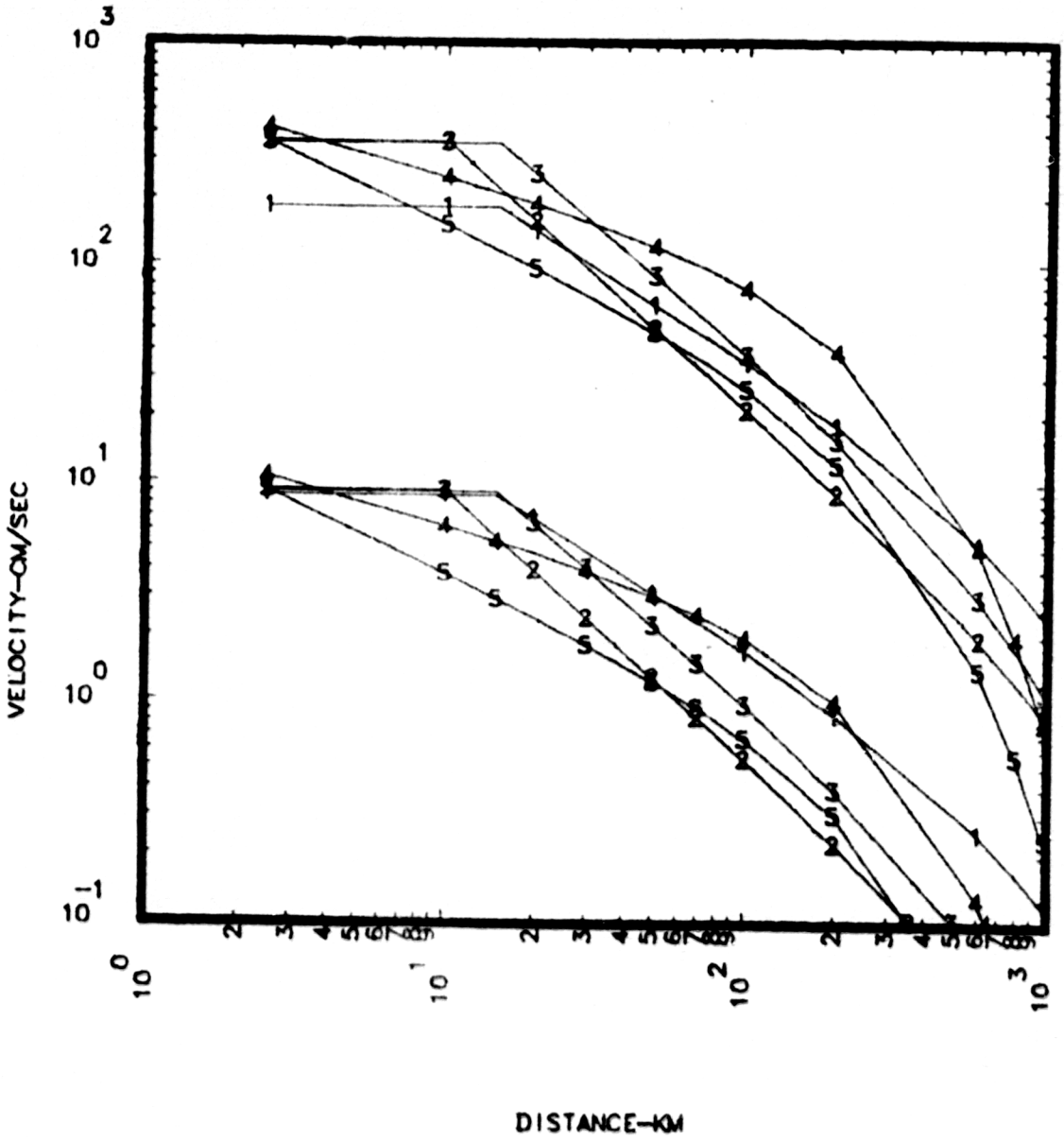


Fig. 2.9 Class 2 Intensity Based - Distance Weighting Velocity Models Selected By GMP.

Plot Symbol	Model Number
2	40
3	35
4	46
5	49

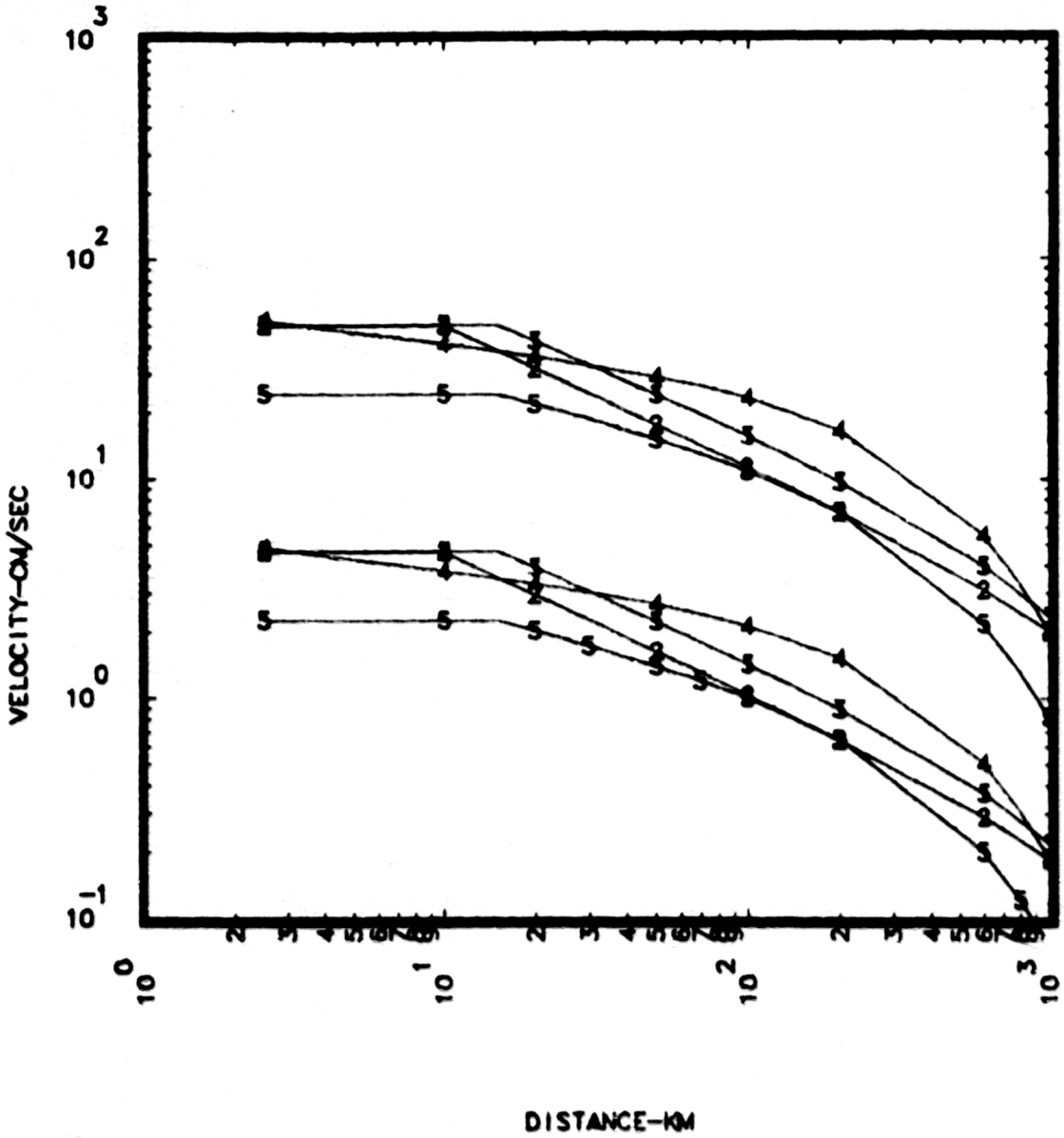


Fig. 2.10 Class 3 Intensity Based - Magnitude Weighting Velocity Models Selected By GMP.

Plot Symbol	Model Number
1	36
2	50
3	52

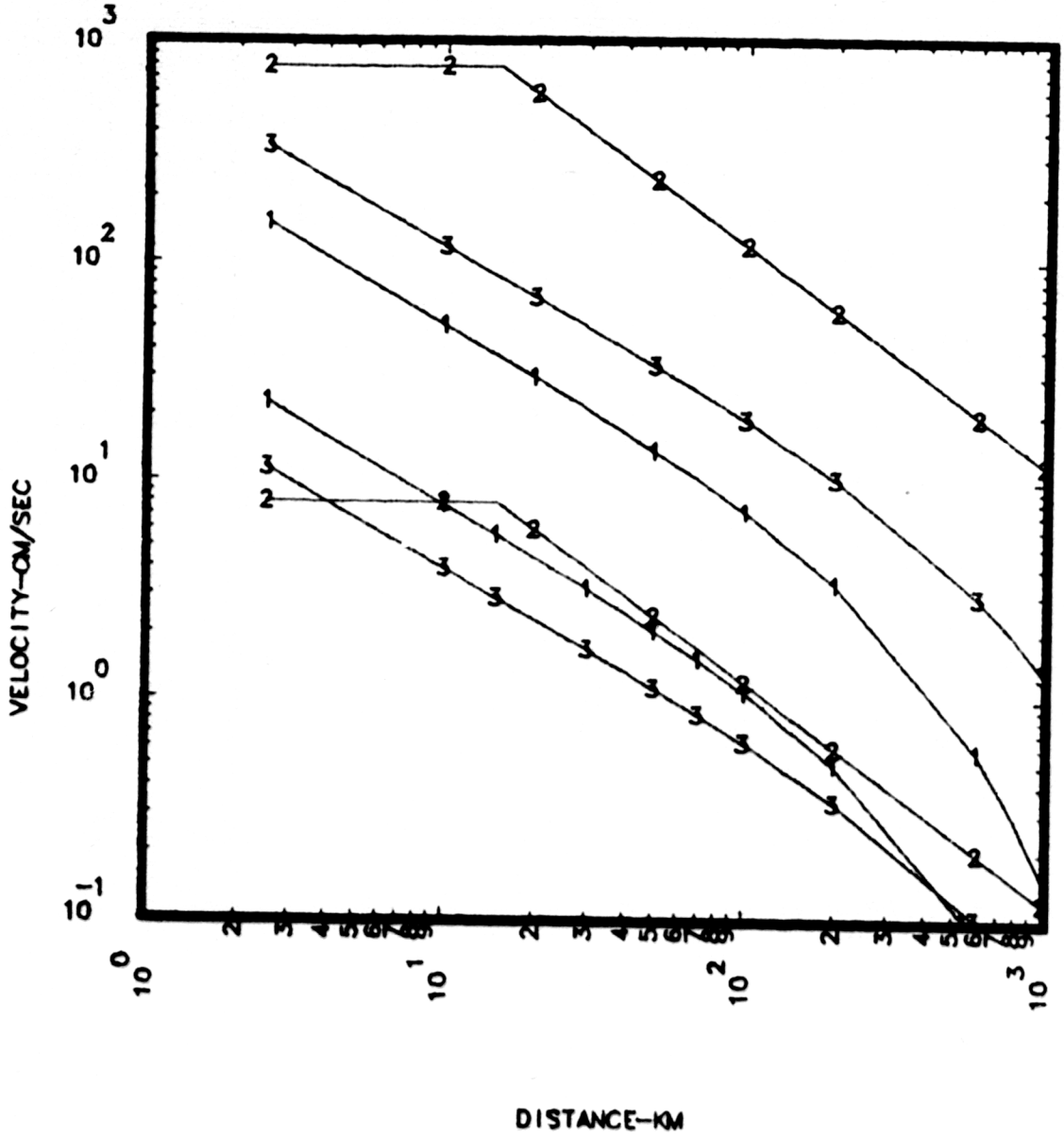


Fig. 2.11 Class 5 Intensity Based - Semi-Empirical Velocity Models Selected By GMP.

Plot Symbol	Model Number
1	51
2	37

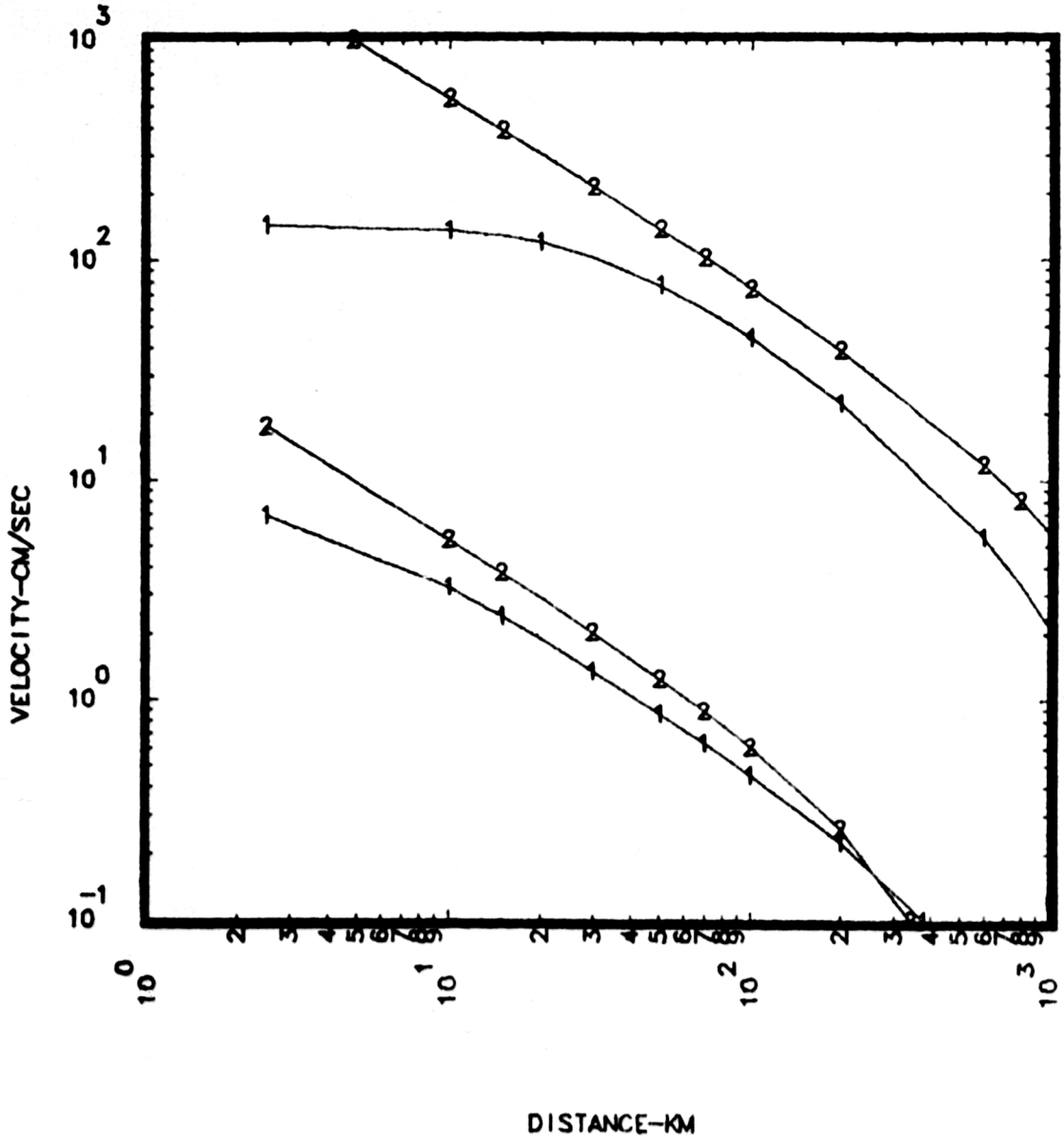


Fig. 2.12 Classes 6 & 7 Direct Velocity Models Chosen By GMP.

Plot Symbol	Model Number
1	71
2	80
3	161
4	170
5	179

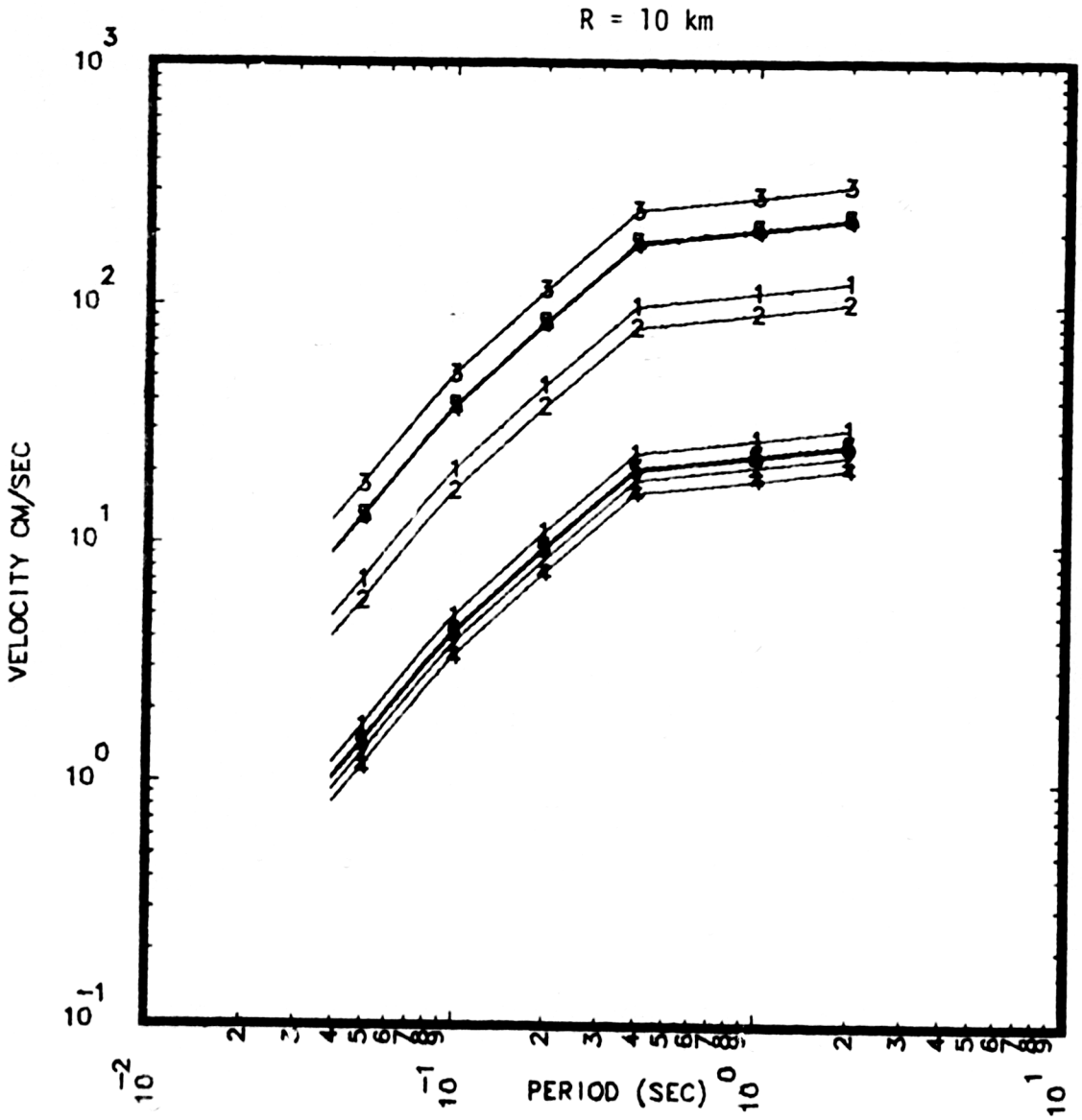


Fig. 2.13a Class RSI Median R.G. 1.60 Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	71
2	80
3	161
4	170
5	179

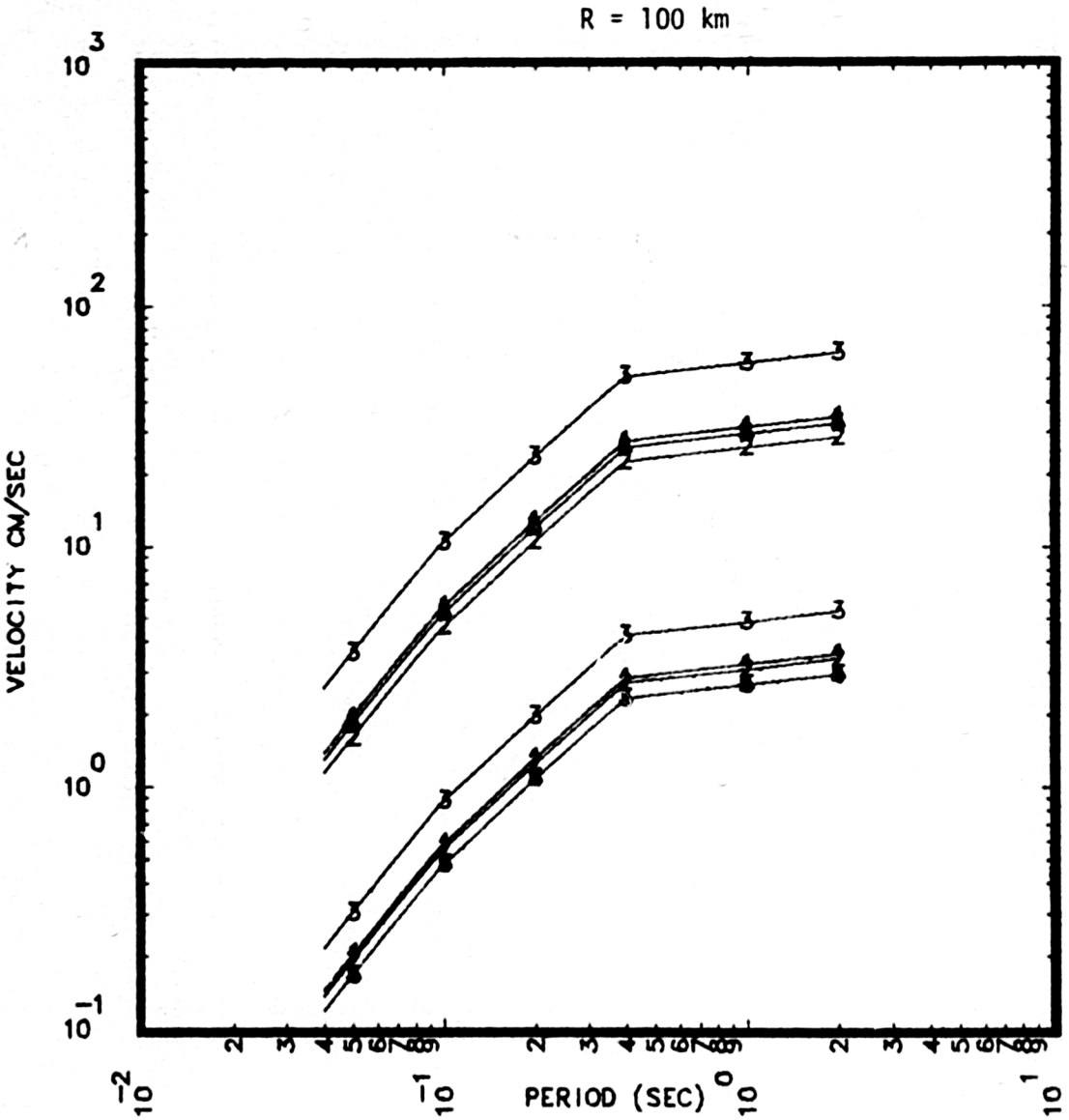


Fig. 2.13b Class RSI Median R.G. 1.60 Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	89
2	98
3	134
4	143
5	152

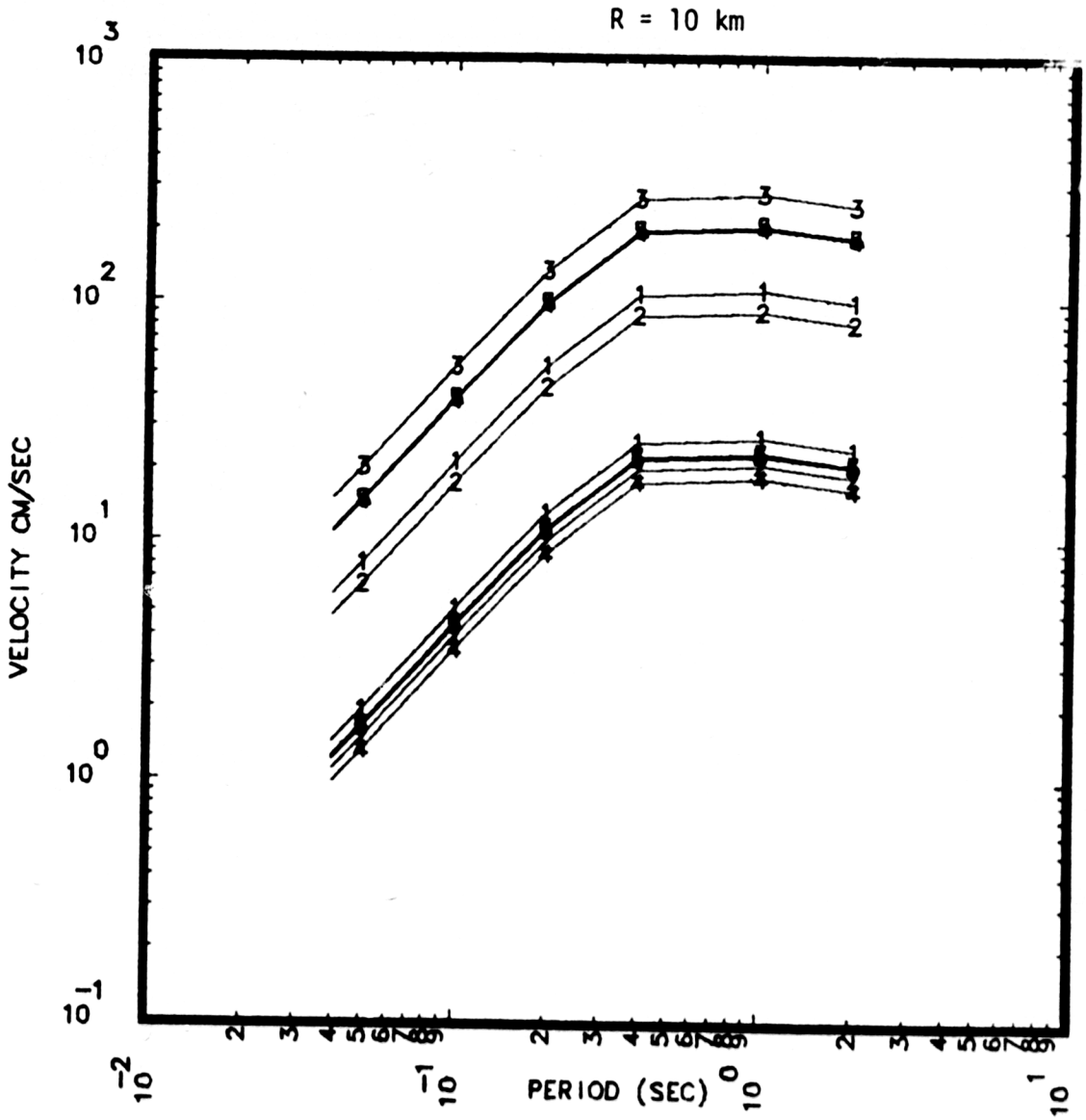


Fig. 2.14a Class RS2 ATC Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	89
2	98
3	134
4	143
5	152

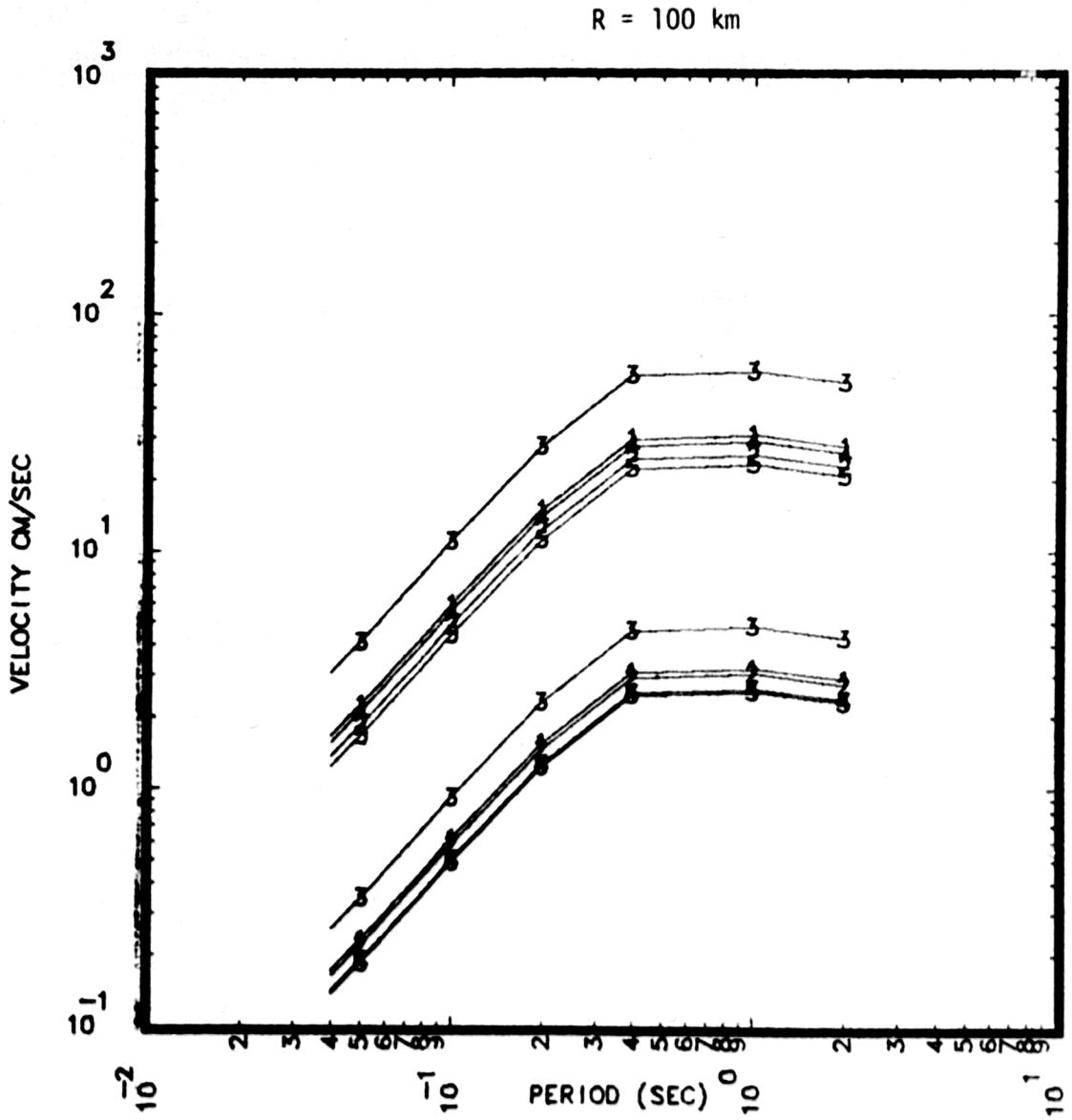


Fig. 2.14b Class RS2 ATC Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	107
2	116
3	188
4	197
5	206

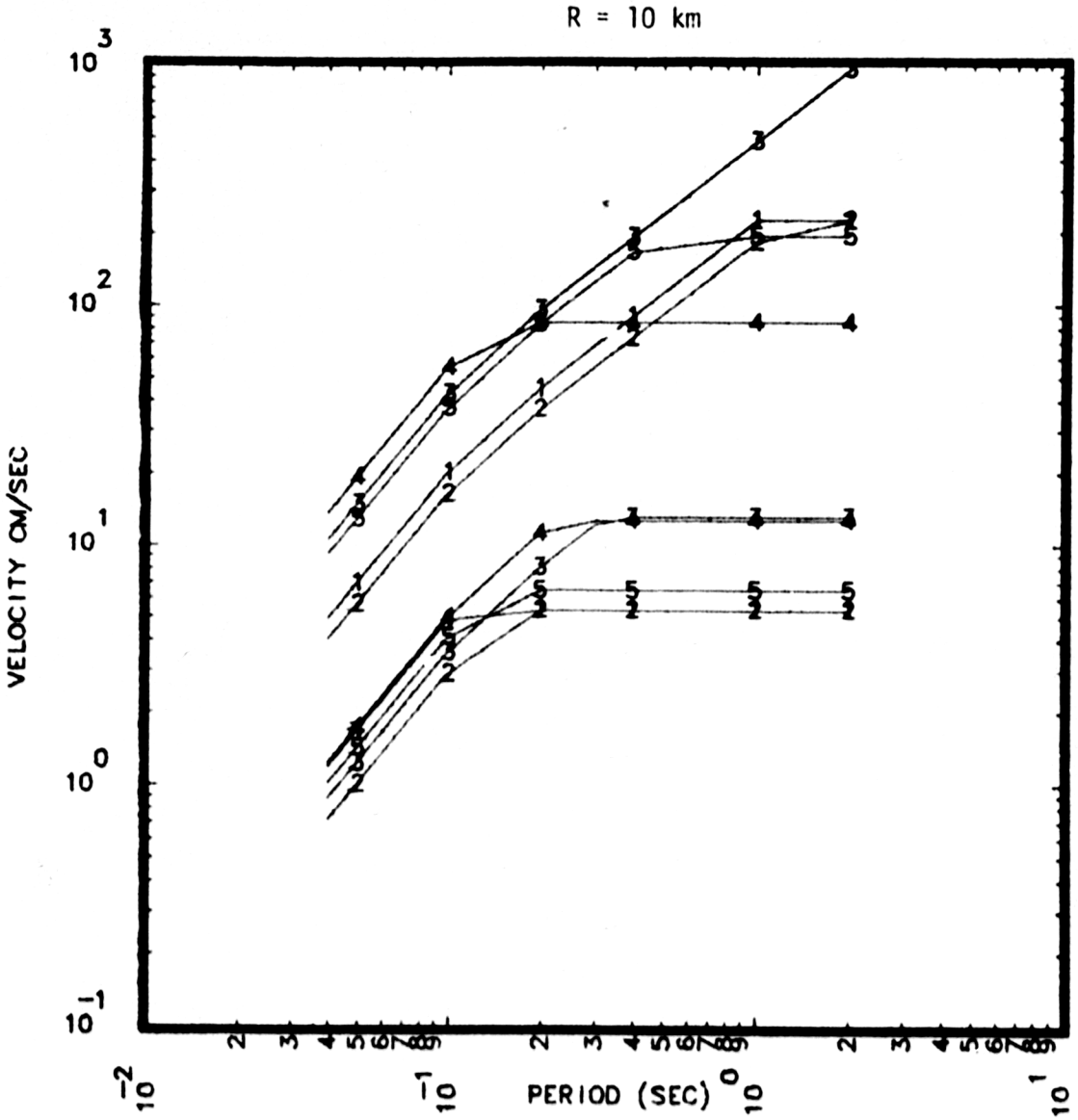


Fig. 2.15a Class RS3 Newmark-Hall Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	107
2	116
3	188
4	197
5	206

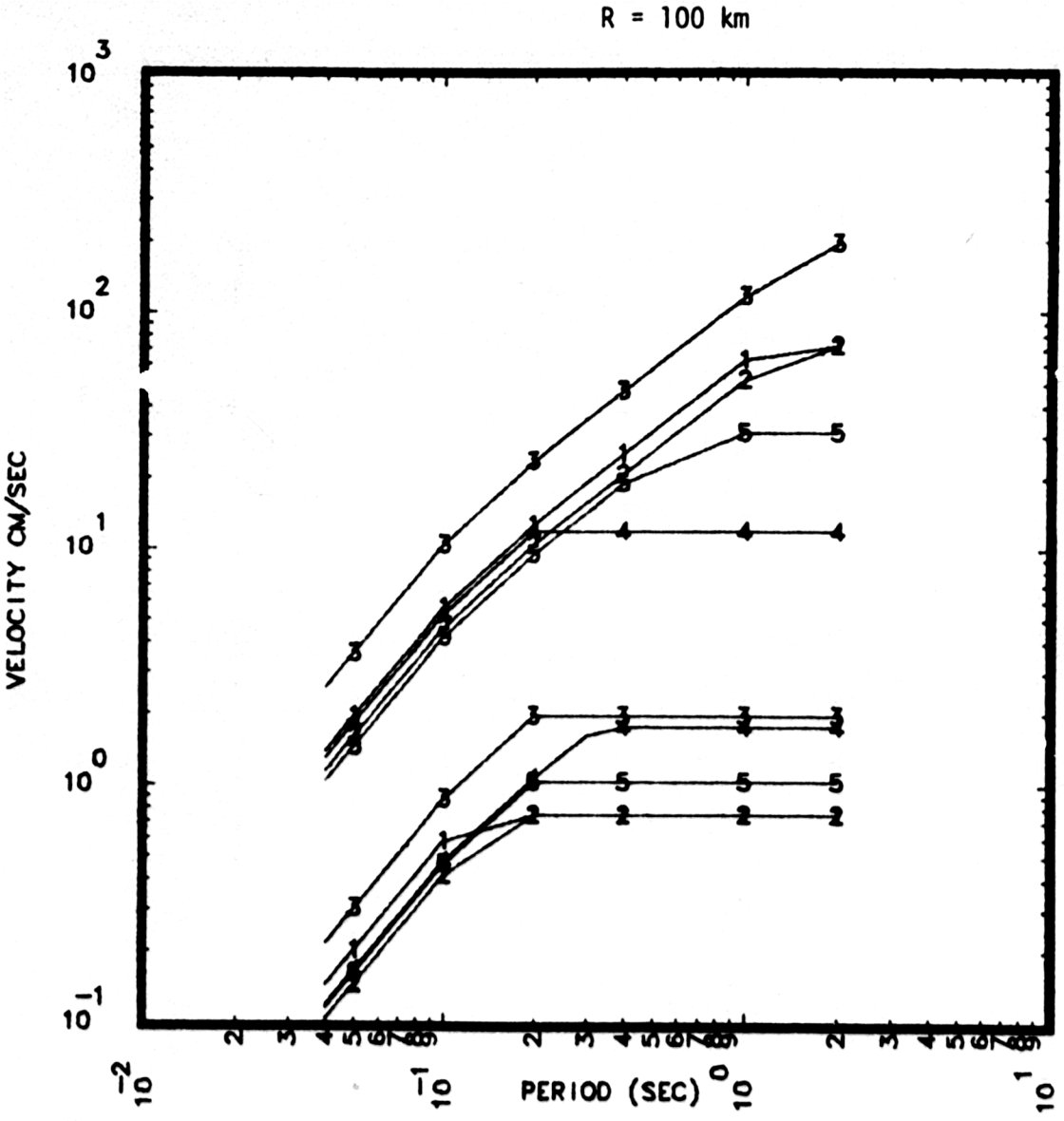


Fig. 2.15b Class RS3 Newmark-Hall Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	53
2	62
3	125

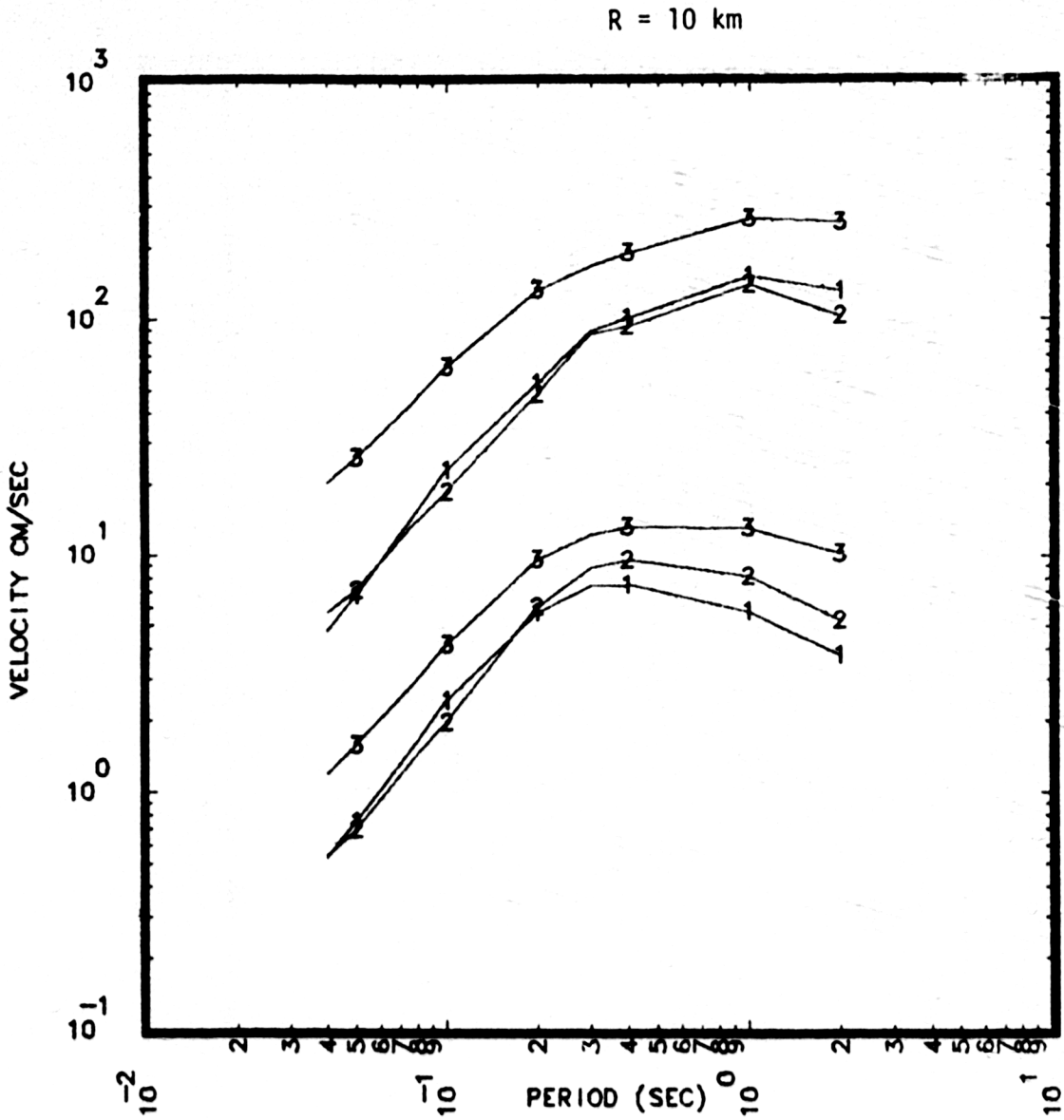


Fig. 2.16a Classes RS4, RS5 & RS6 Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	53
2	62
3	125

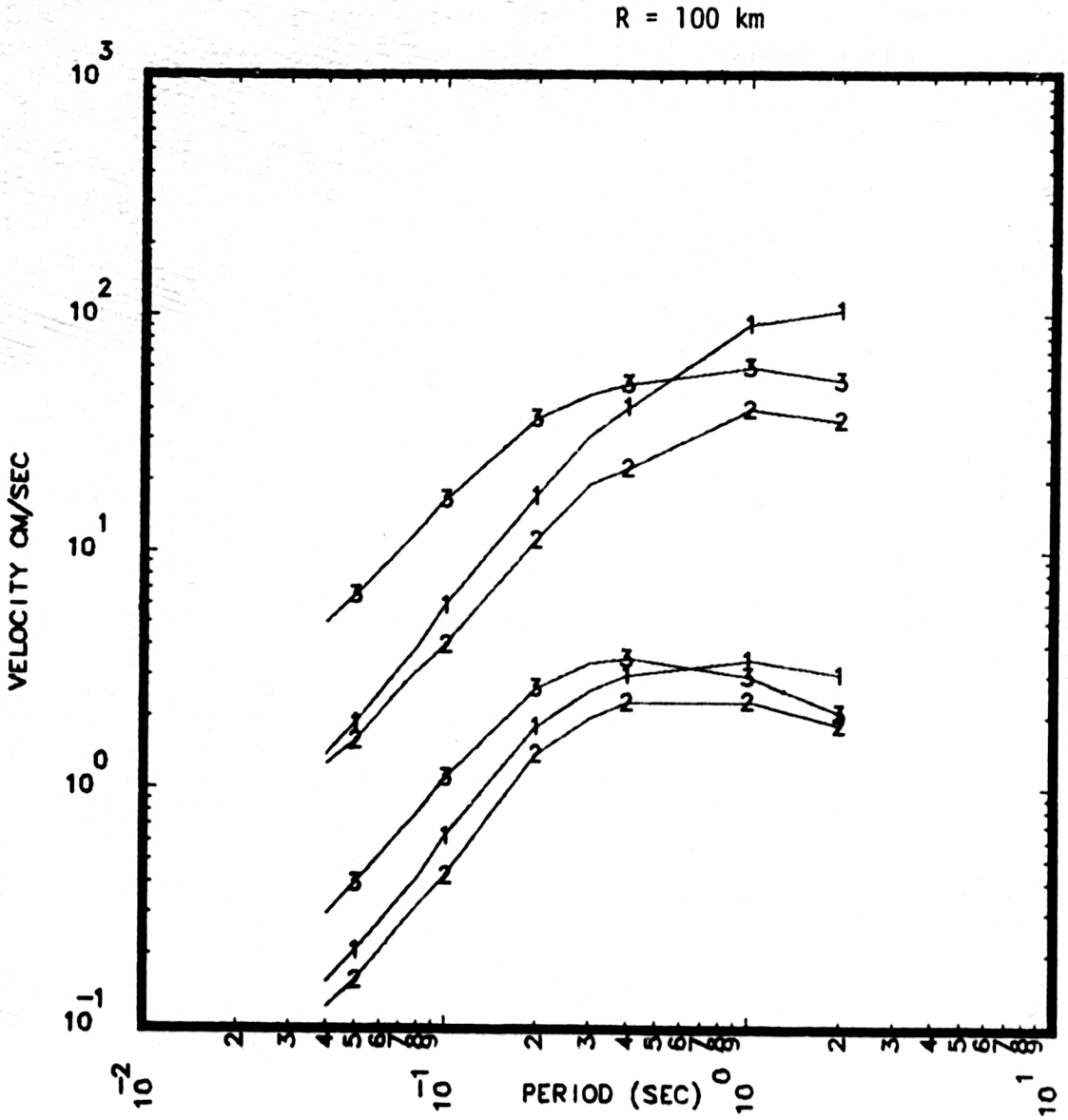


Fig. 2.16b Classes RS4, RS5 & RS6 Spectral Models Chosen By GMP.

Plot Symbol	Model Number
1	5
2	7
3	24
4	25
5	27
6	28

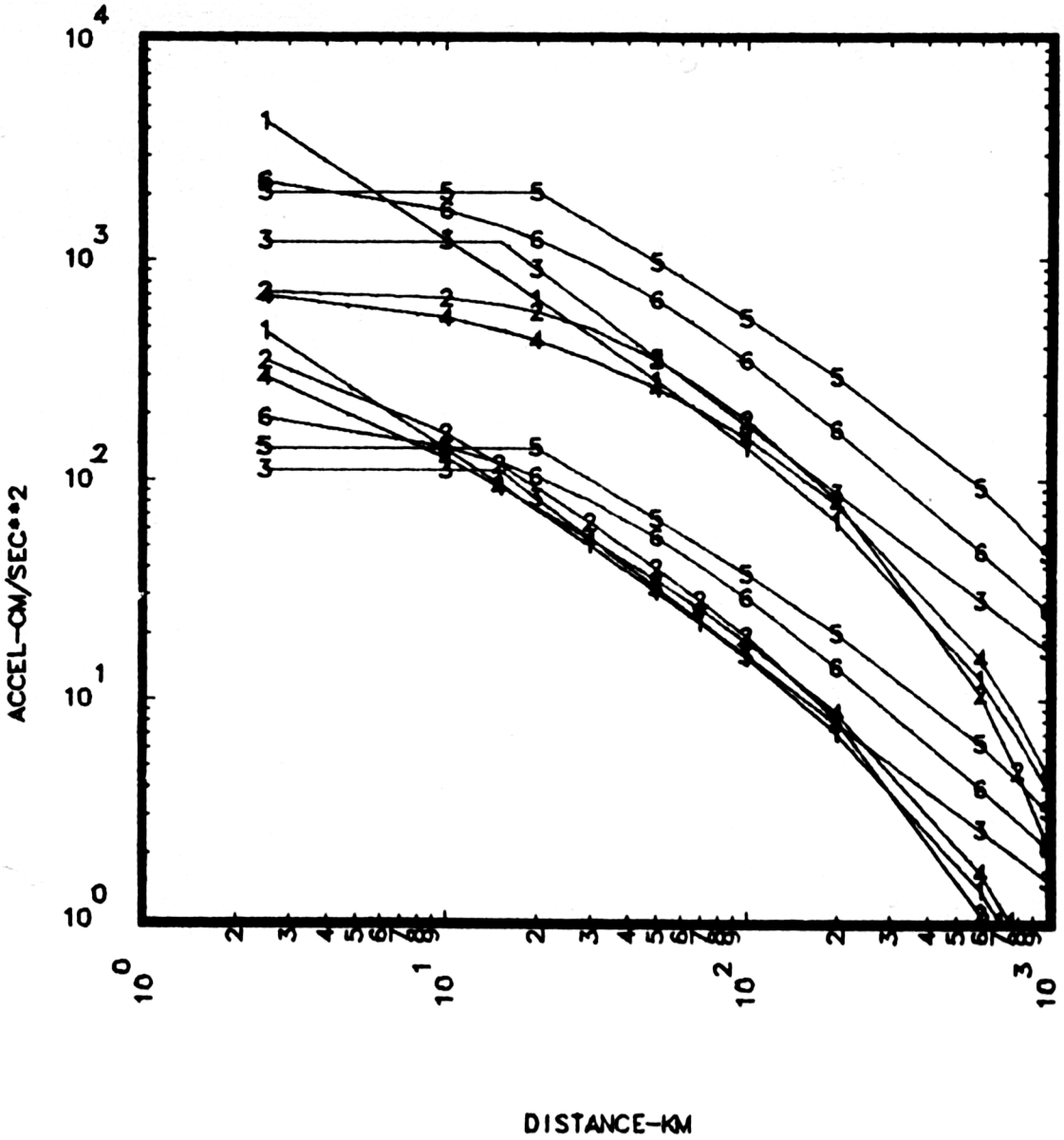


Fig. 2.17 Best Estimate Acceleration Models Selected By GMP.

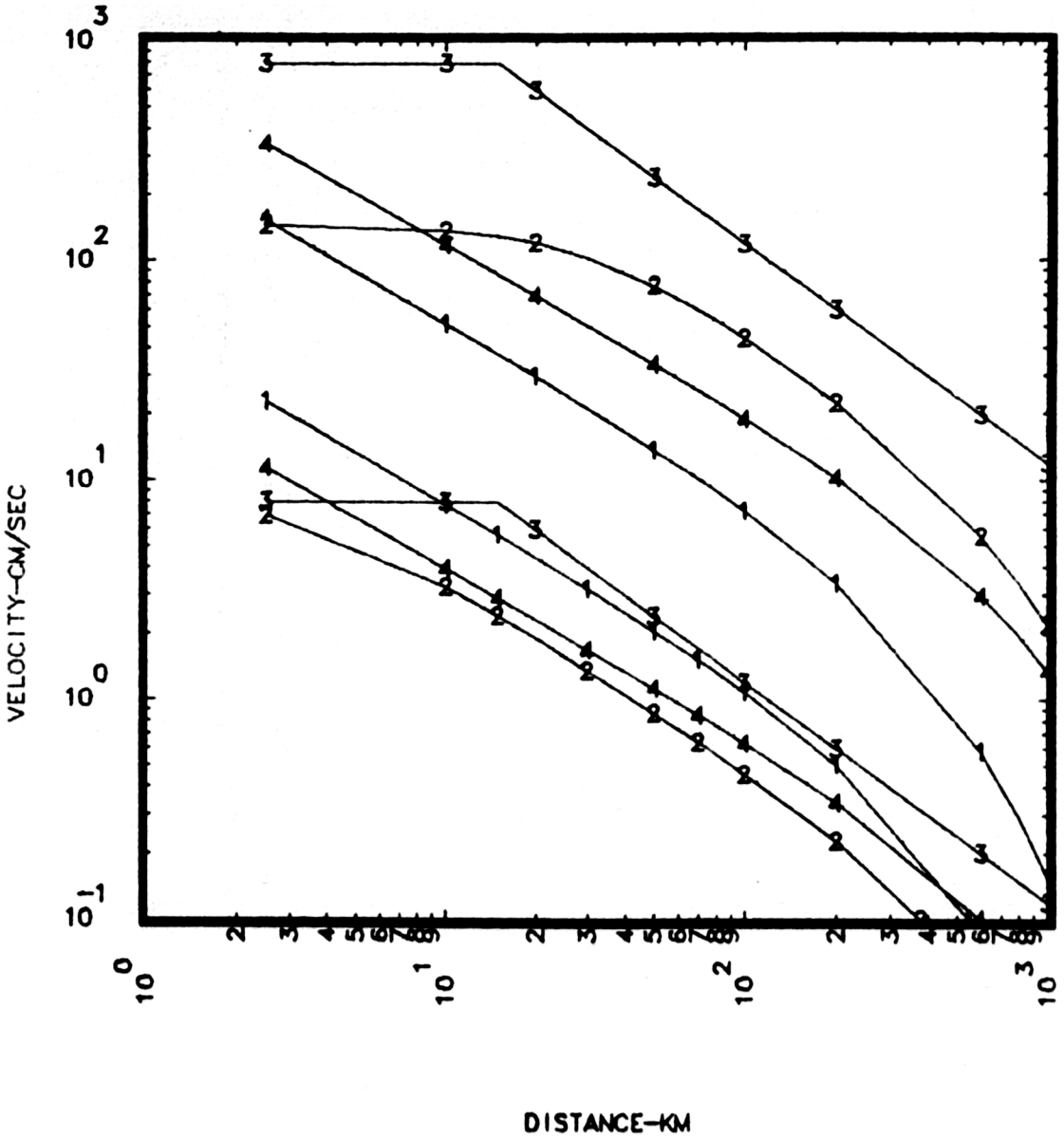


Fig. 2.18 Best Estimate Velocity Models Selected By GMP.

Plot Symbol	Model Number
1	107
2	116
3	125
4	134
5	152
6	170

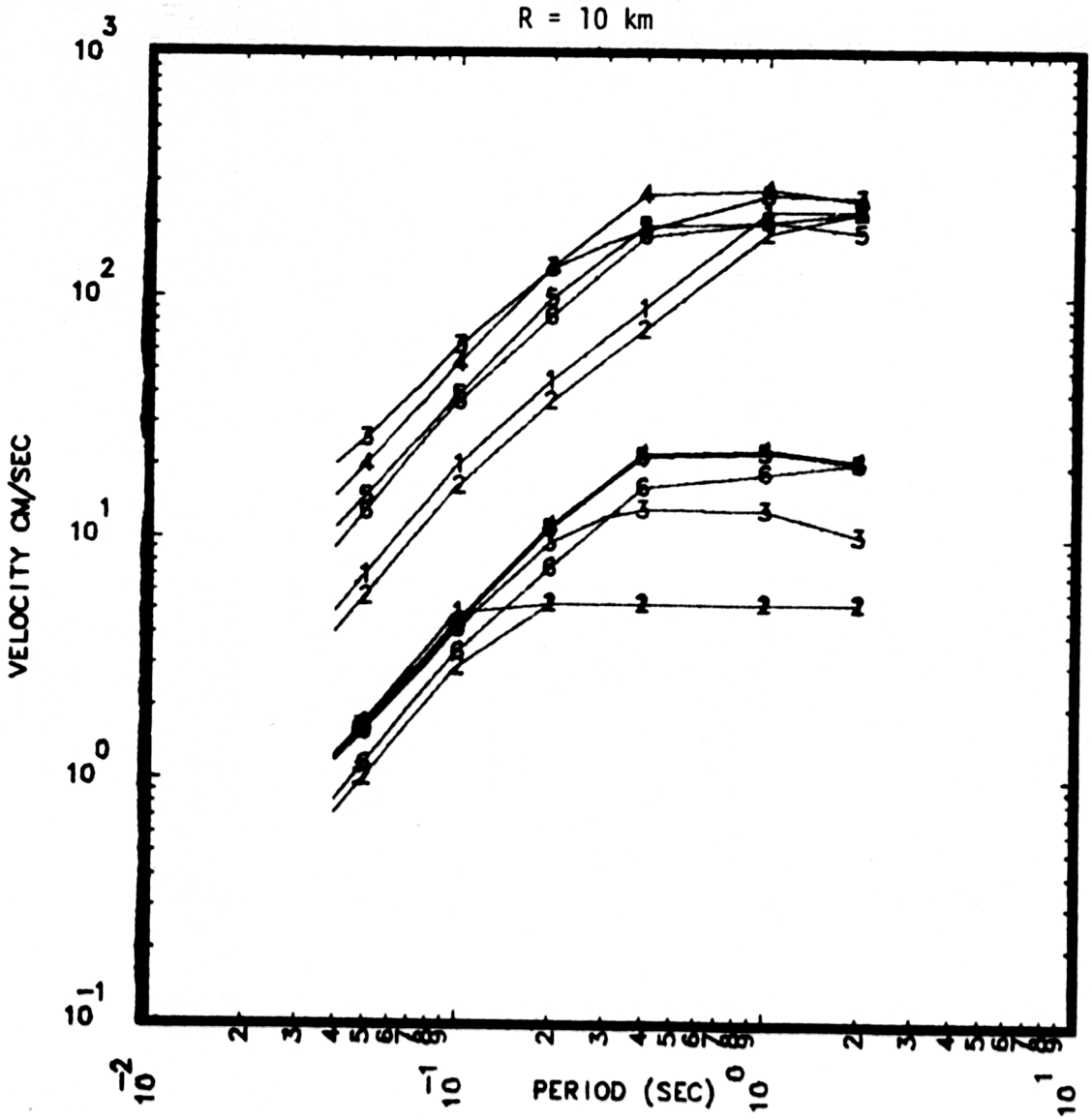


Fig. 2.19a Best Estimate Spectral Models Selected By GMP.

Plot Symbol	Model Number
1	107
2	116
3	125
4	134
5	152
6	170

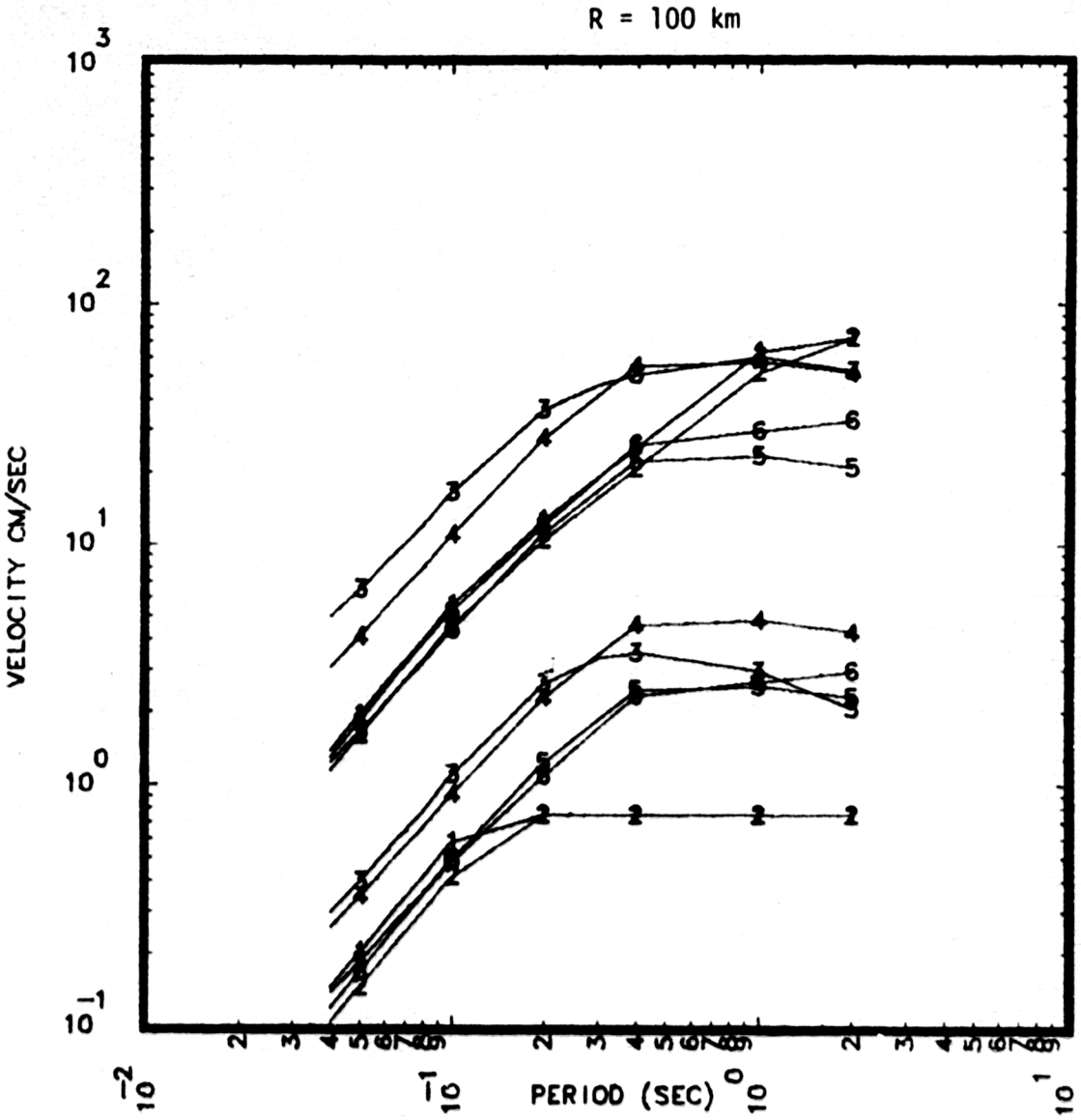


Fig. 2.19b Best Estimate Spectral Models Selected By GMP.

3. IMPACT OF THE GROUND MOTION MODELS ON THE HAZARD

3.1 Introduction

The impact of the ground motion models on the hazard has been described and analyzed in detail in the Interim Report. However, the availability of an additional set of models provided to us by expert 2 made it necessary to update the analysis.

We selected two sites, Braidwood and Millstone, very different in their seismic environment, and three seismicity experts, experts 1, 6 and 12 in an effort to cover the widest possible spectrum of cases with a limited computational effort.

- For each site and each of the 3 seismicity experts, we show the hazard calculated with the B.E. ground motion model of each of the 5 ground motion experts for the acceleration and for the pseudo velocity spectra.

- Then for each ground motion expert we show the hazard calculated with each one of the ground motion models provided, considering that all the other parameters (seismicity, maps) are random.

- Finally, for these two sites we give the hazard calculated by combining all the ground motion experts, but still with only the 3 seismicity experts' input selected above and letting all the random parameters free of varying.

3.2 Best Estimate (B.E.) Ground Motion Models

The hazard was calculated for the two example sites and 3 seismicity experts by assigning to each random variable its best estimate value. These calculations allow to make relative comparisons between the various ground motion models for a given set of maps and for known set values of the other parameters. Figures 3.1 to 3.3 show the hazard calculated for the example

site Braidwood, for the seismicity experts 1, 6 and 12 respectively. These figures show 5 curves. Each one of these 5 curves has been obtained by using only the B.E. ground motion model of the ground motion expert whose index is associated with the curve. Figures 3.4 to 3.6 is the same as above applied to the sample site Millstone. Note that these calculations were made prior to the ground motion feedback meeting, before the model #27 was updated. Since model 27 was chosen as B.E. by expert number 5, we expect the final analysis to provide lower estimates of the hazard as the ones shown here.

Figures 3.7 and 3.8 show the B.E. hazard at the two sample sites, combined over the 5 ground motion experts.

3.3 Other Ground Motion Models impact on the Hazard

In order to analyze the actual effect of every ground motion model on the final hazard, we calculate the hazard by considering the uncertainty on all the parameters of the analysis but in the ground motion model. That is, we fix the choice of a ground motion model and let all other random variable be simulated.

Fig. 3.9 to 3.18 show the 50th percentile hazard curves grouped by ground motion experts. For instance, Fig. 3.9 shows the 50th hazard curve for the six ground motion models selected by ground motion expert 1.

3.4 Case of the hazard in terms of P.S.V. spectra

Because of the added dimension due to the consideration of frequency, only the simplest cases are considered here. The B.E. hazard was calculated at the two sample sites, combined over the three seismicity experts 1, 6 and 12. The Figures 3.19 and 3.20 show the hazard, for the two sites for a 1000 yr return period. Each figure includes 3 curves relative to the 3 seismicity experts. No individual calculations, per spectral model, are shown here, but results of such calculations were shown at the meeting. The results of these calculations emphasize the effect of each one of the spectral models on the final hazard.

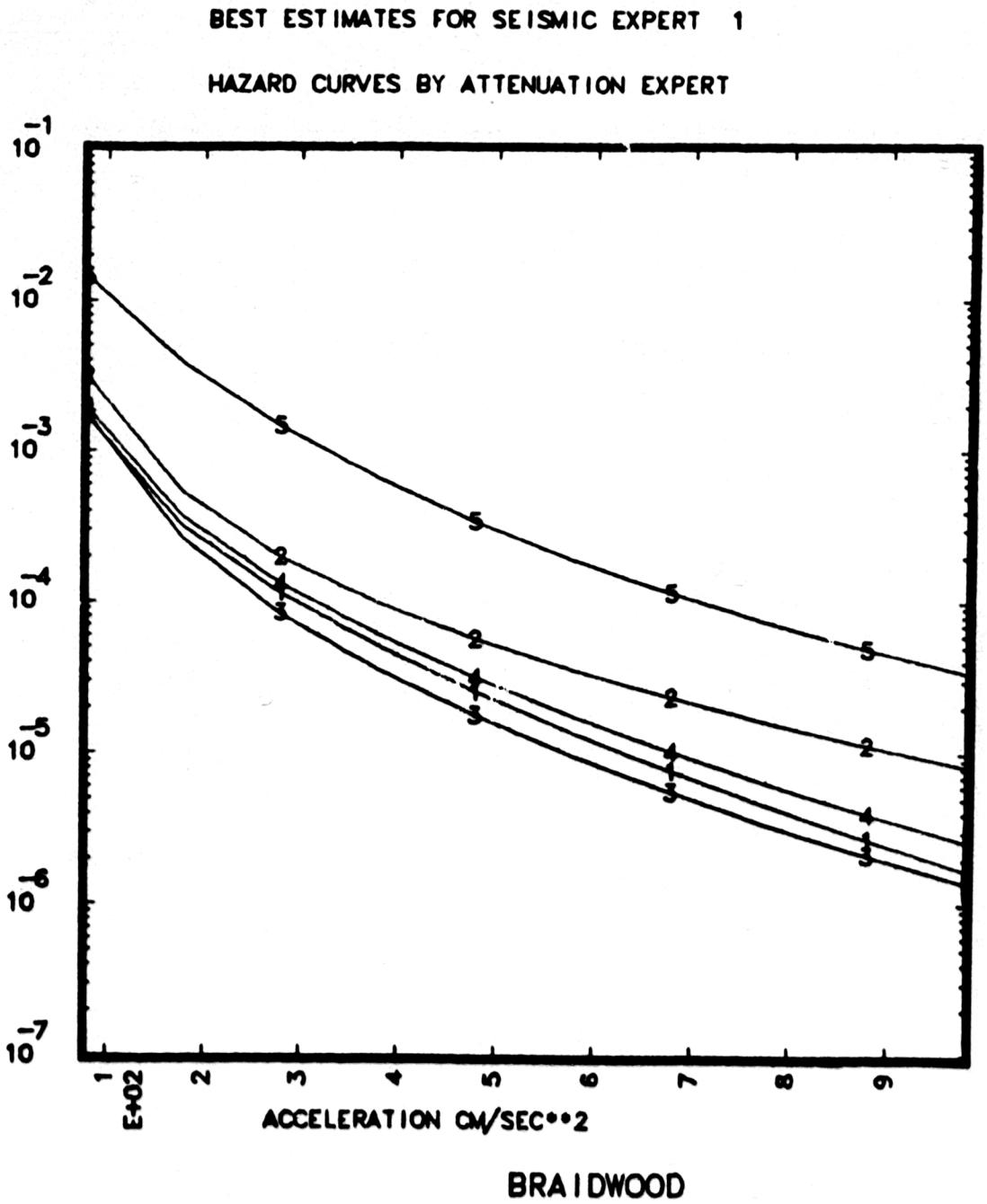

BEST ESTIMATES FOR SEISMIC EXPERT 1
HAZARD CURVES BY ATTENUATION EXPERT
10^-1
10^-2
10^-3
10^-4
10^-5
10^-6
10^-7
1
E+02
2
3
4
5
6
7
8
9
ACCELERATION CM/SEC**2
BRAIDWOOD

Fig. 3.1

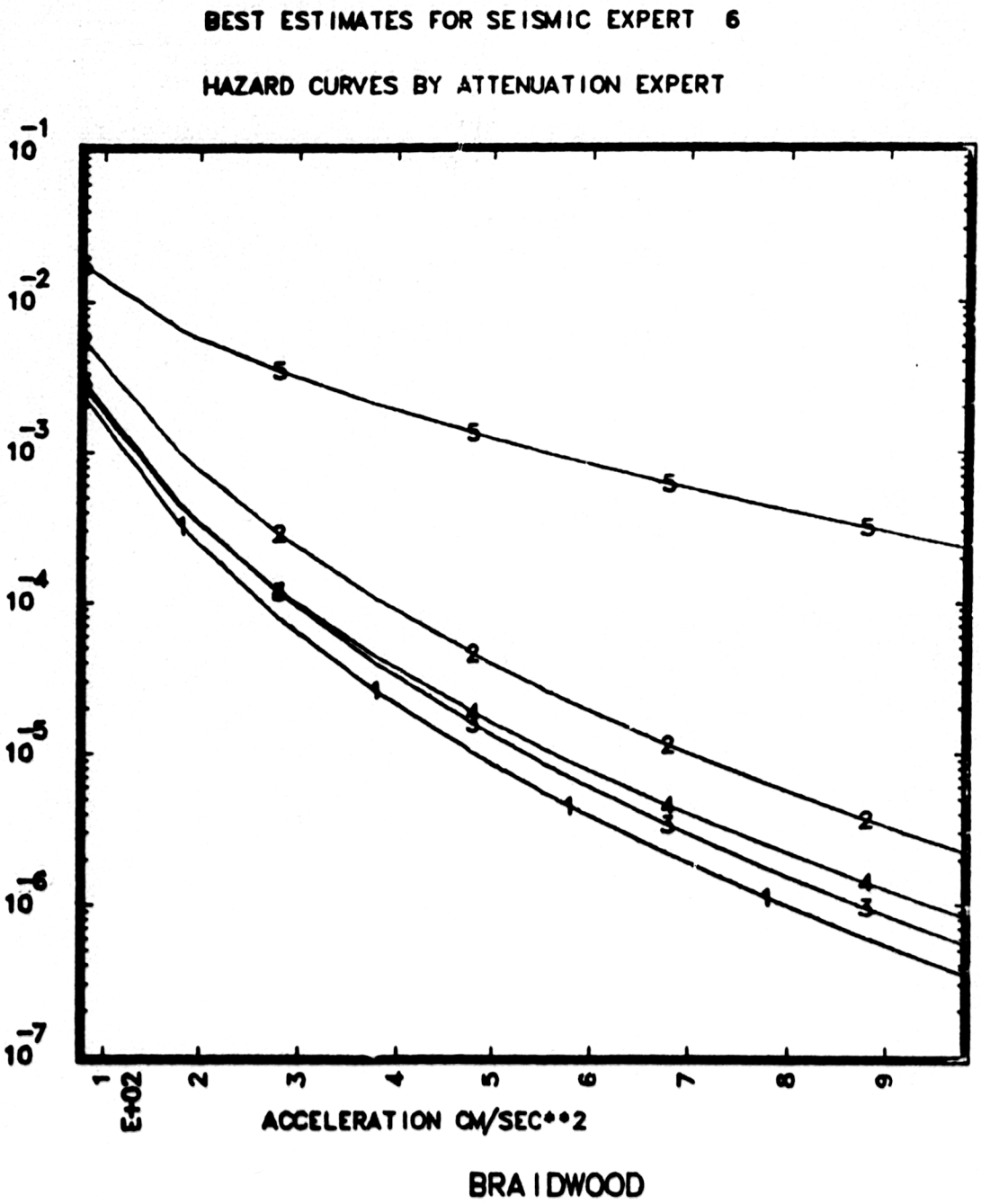
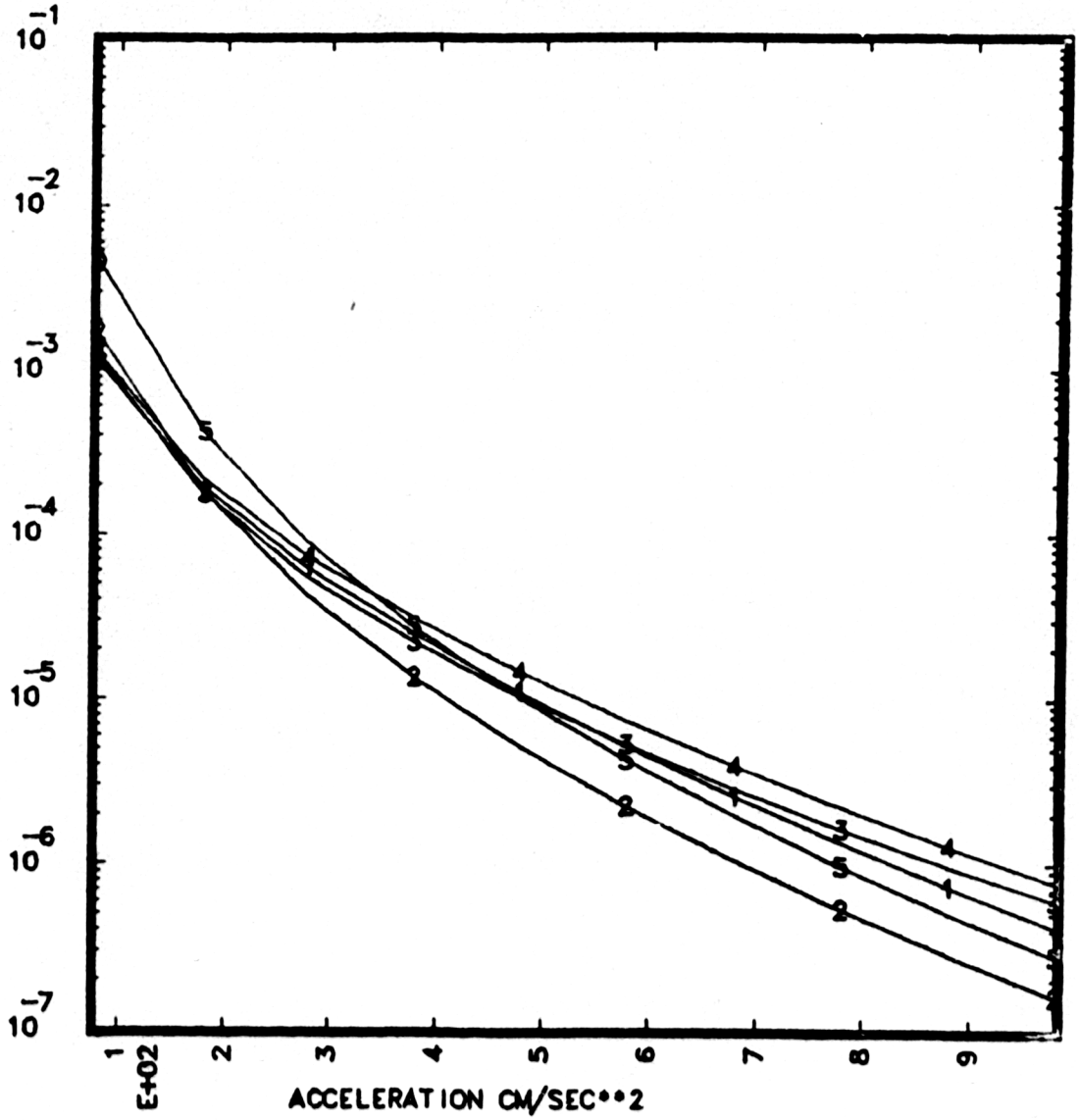

BEST ESTIMATES FOR SEISMIC EXPERT 6
HAZARD CURVES BY ATTENUATION EXPERT
10^-1
10^-2
10^-3
10^-4
10^-5
10^-6
10^-7
1
E+02
2
3
4
5
6
7
8
9
ACCELERATION CM/SEC**2
BRAIDWOOD

Fig. 3.2

BEST ESTIMATES FOR SEISMIC EXPERT 12

HAZARD CURVES BY ATTENUATION EXPERT

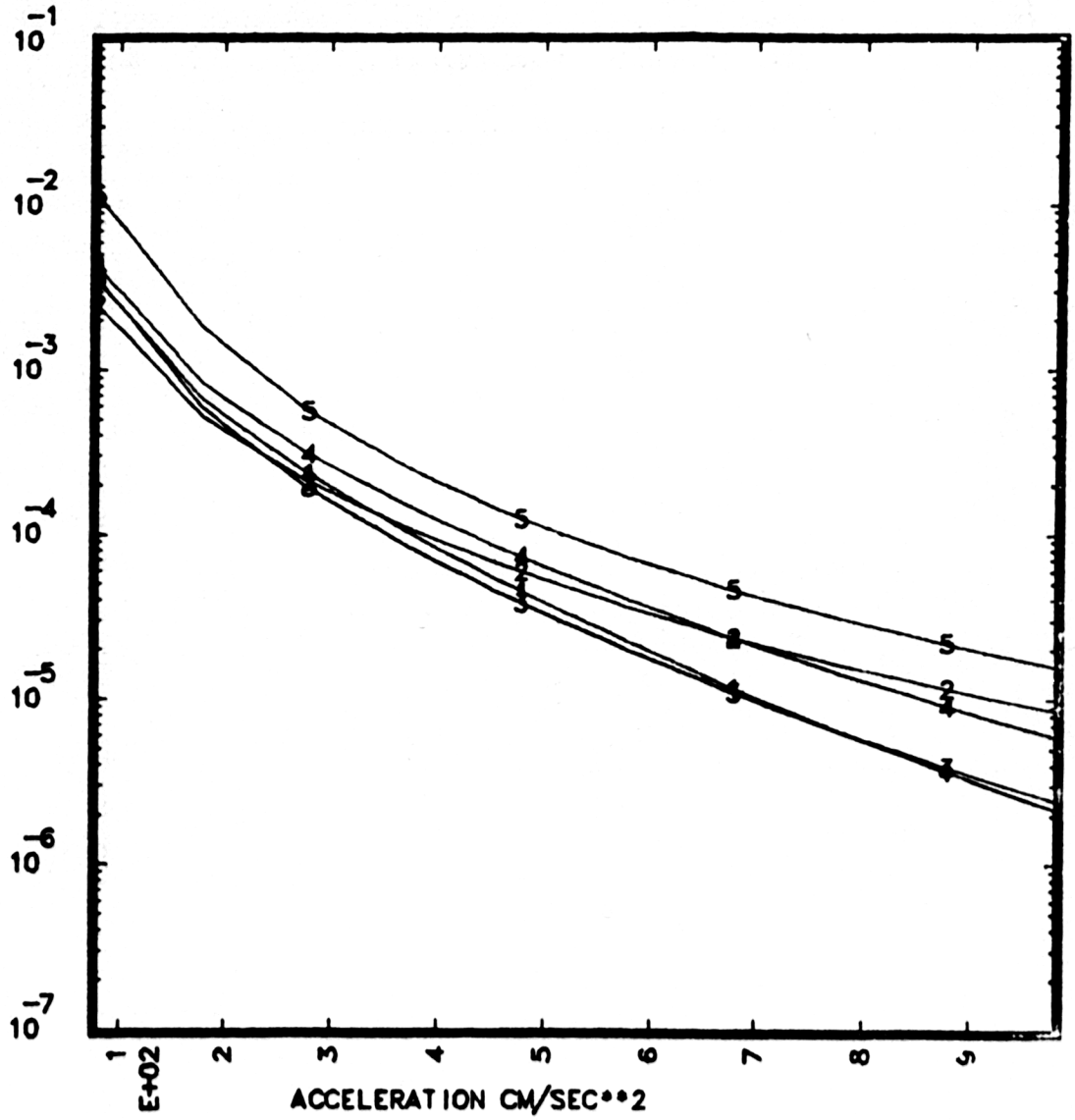


BRAIDWOOD

Fig. 3.3

BEST ESTIMATES FOR SEISMIC EXPERT 1

HAZARD CURVES BY ATTENUATION EXPERT

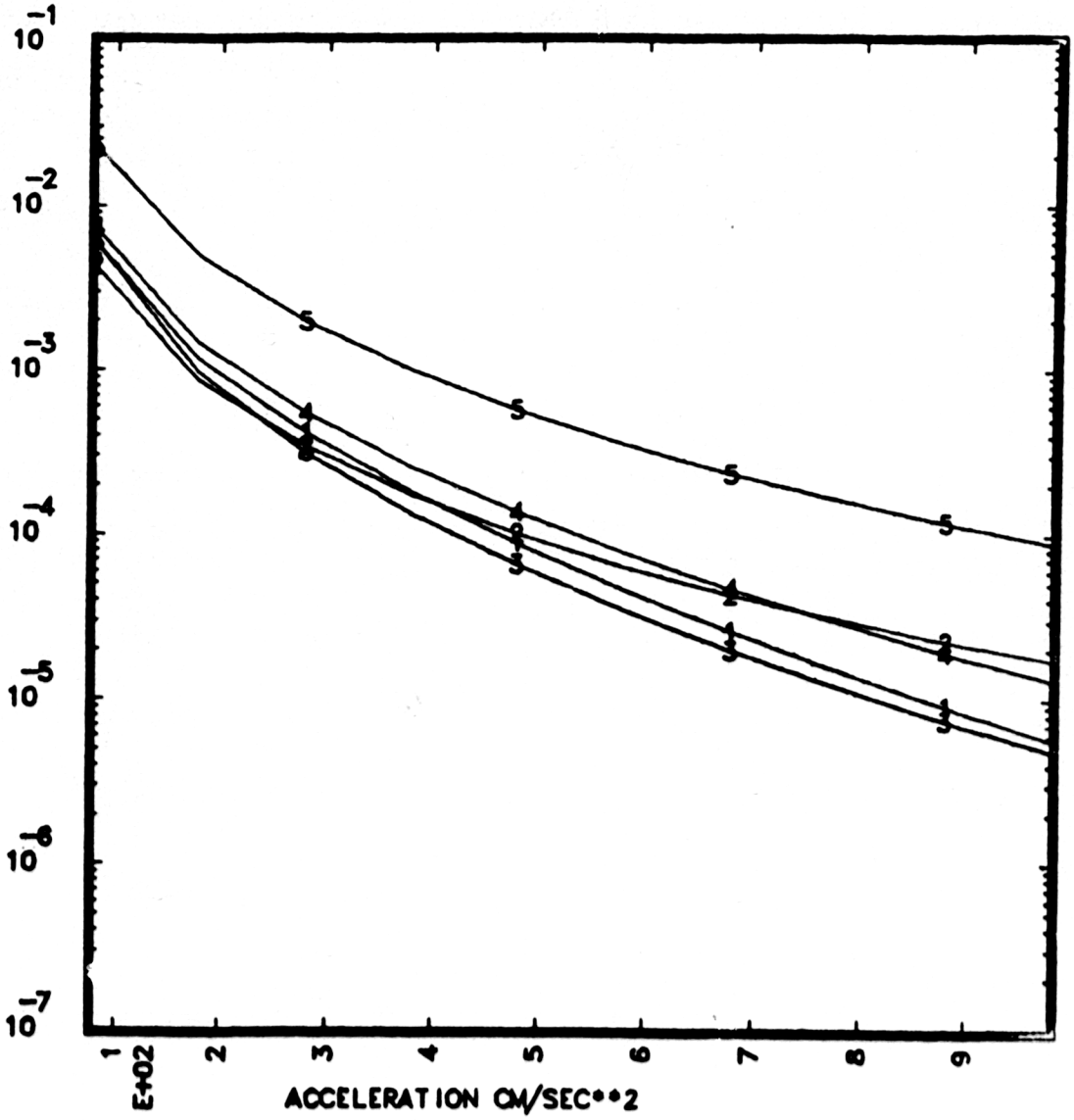


MILLSTONE

Fig. 3.4

BEST ESTIMATES FOR SEISMIC EXPERT 6

HAZARD CURVES BY ATTENUATION EXPERT

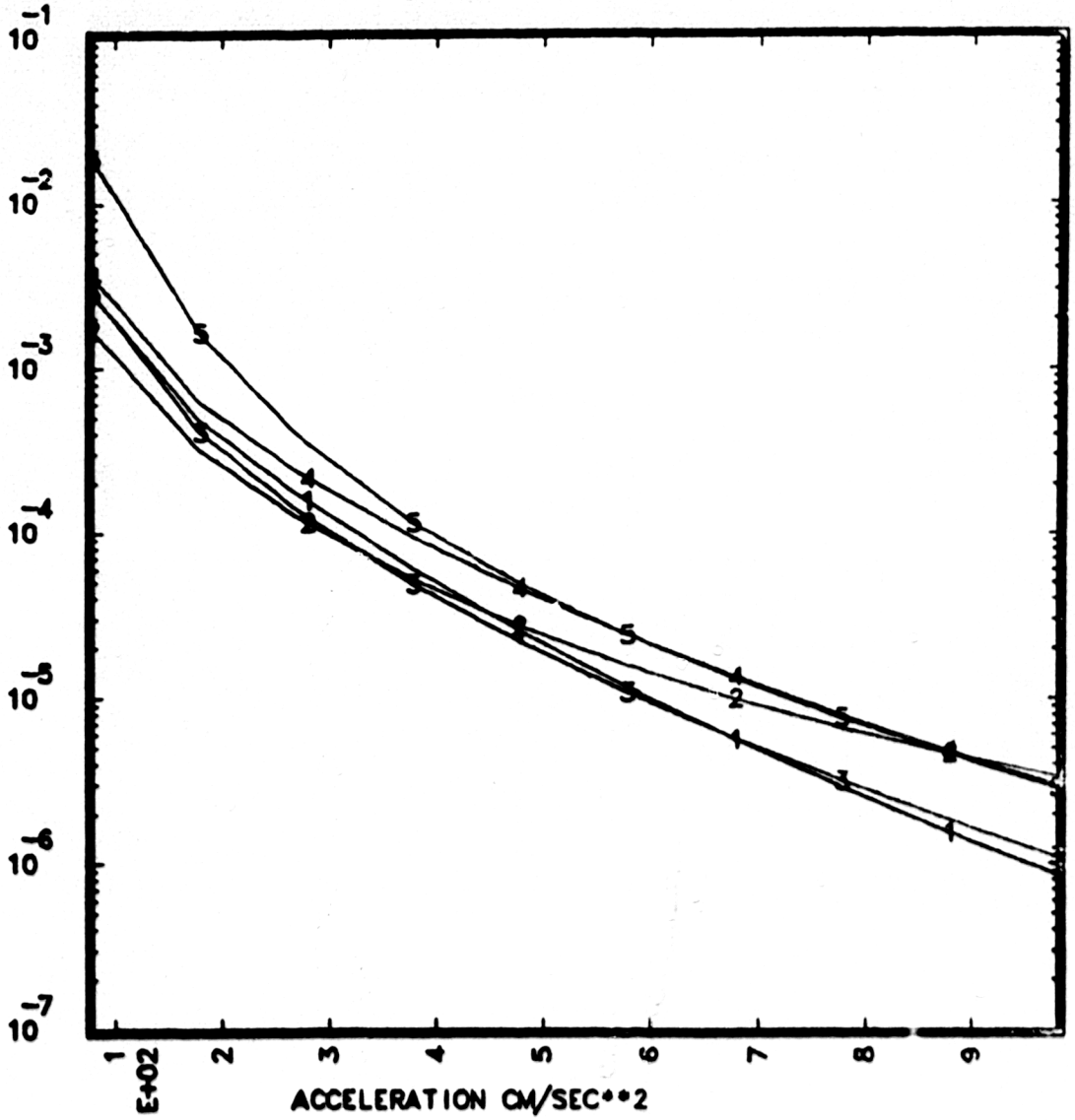


MILLSTONE

Fig. 3.5

BEST ESTIMATES FOR SEISMIC EXPERT 12

HAZARD CURVES BY ATTENUATION EXPERT



MILLSTONE

Fig. 3.6

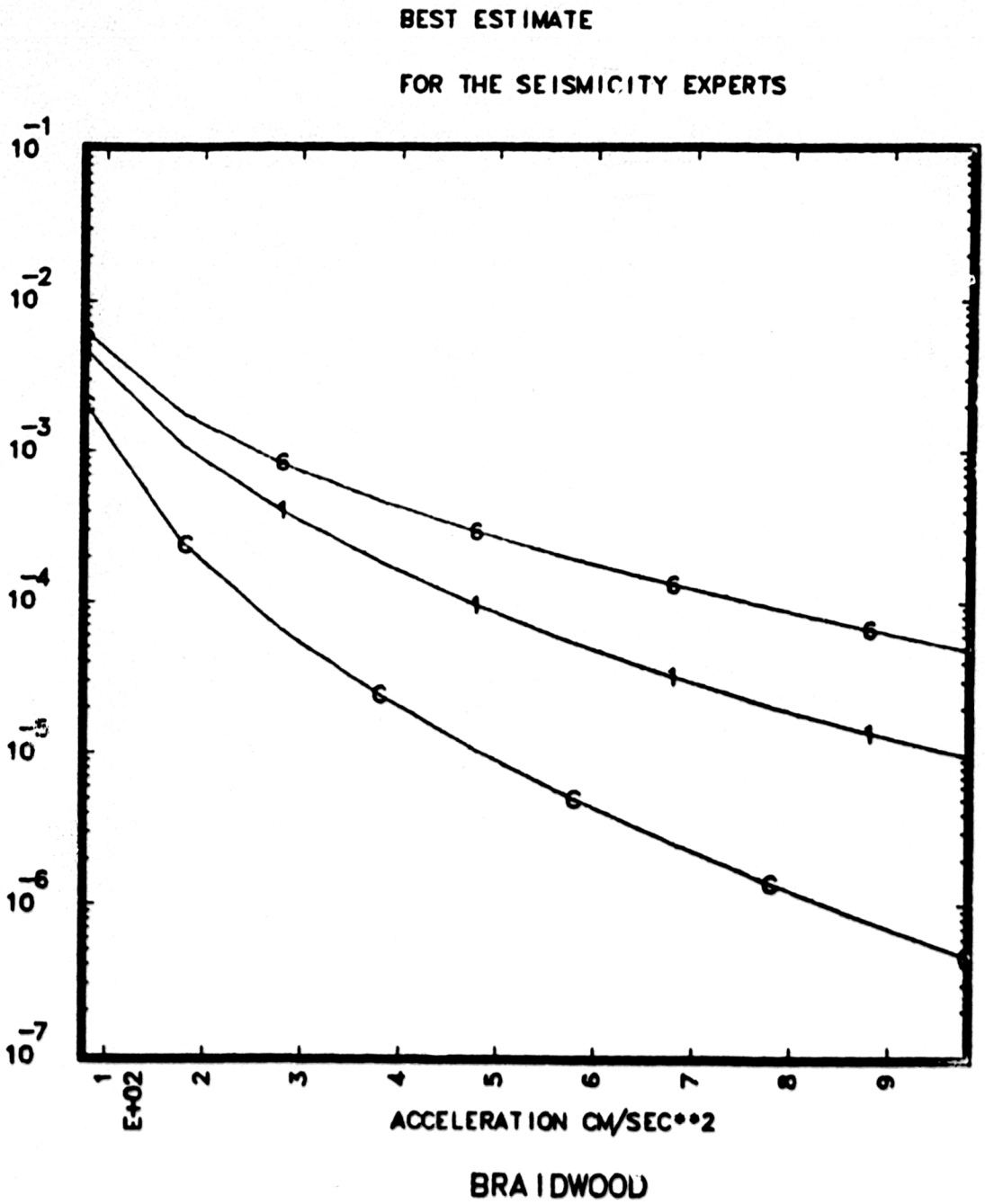


Fig. 3.7 Hazard Combined Over The 5 Ground Motion Experts.

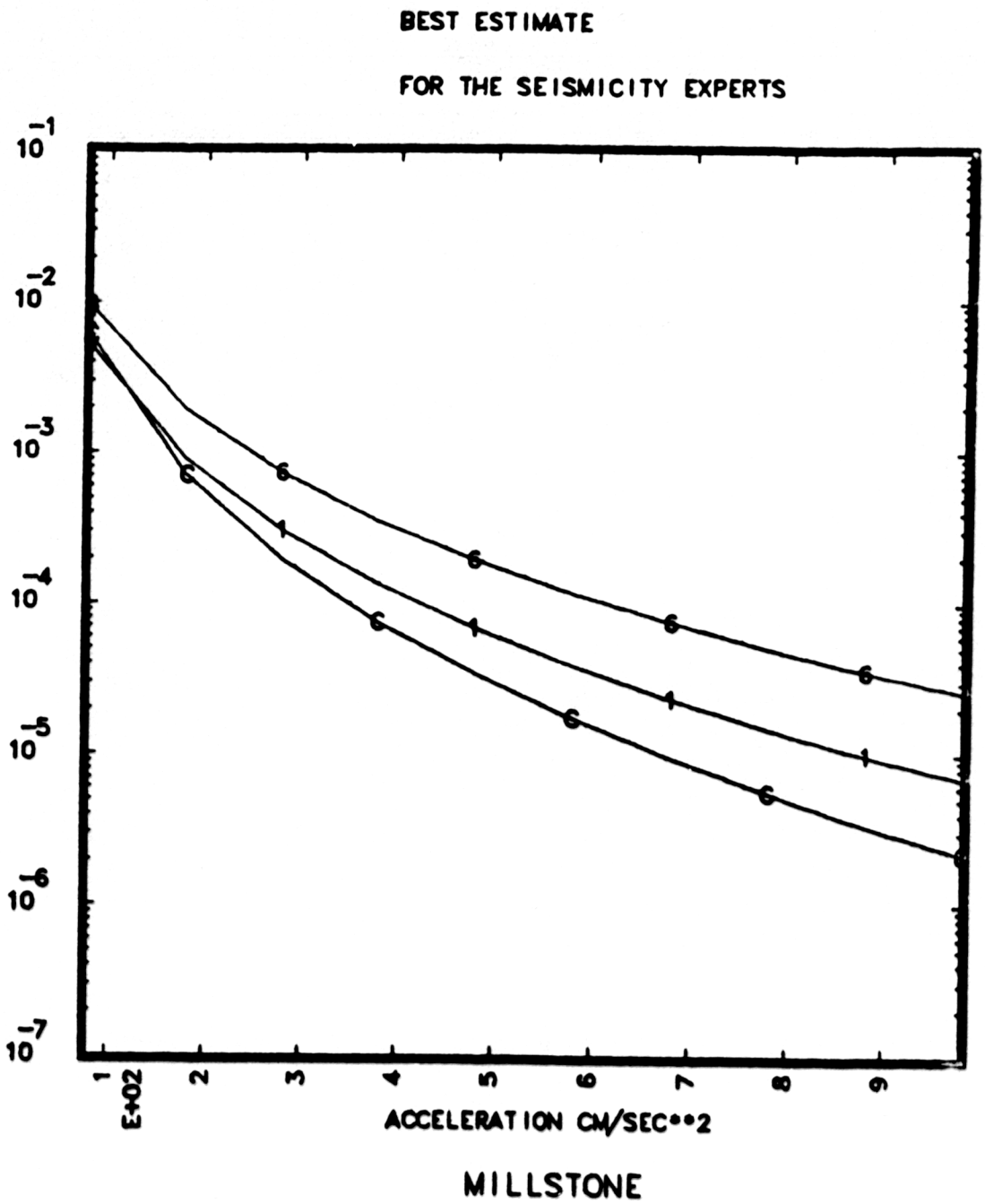


Fig. 3.8 Hazard Combined Over The 5 Ground Motion Experts.

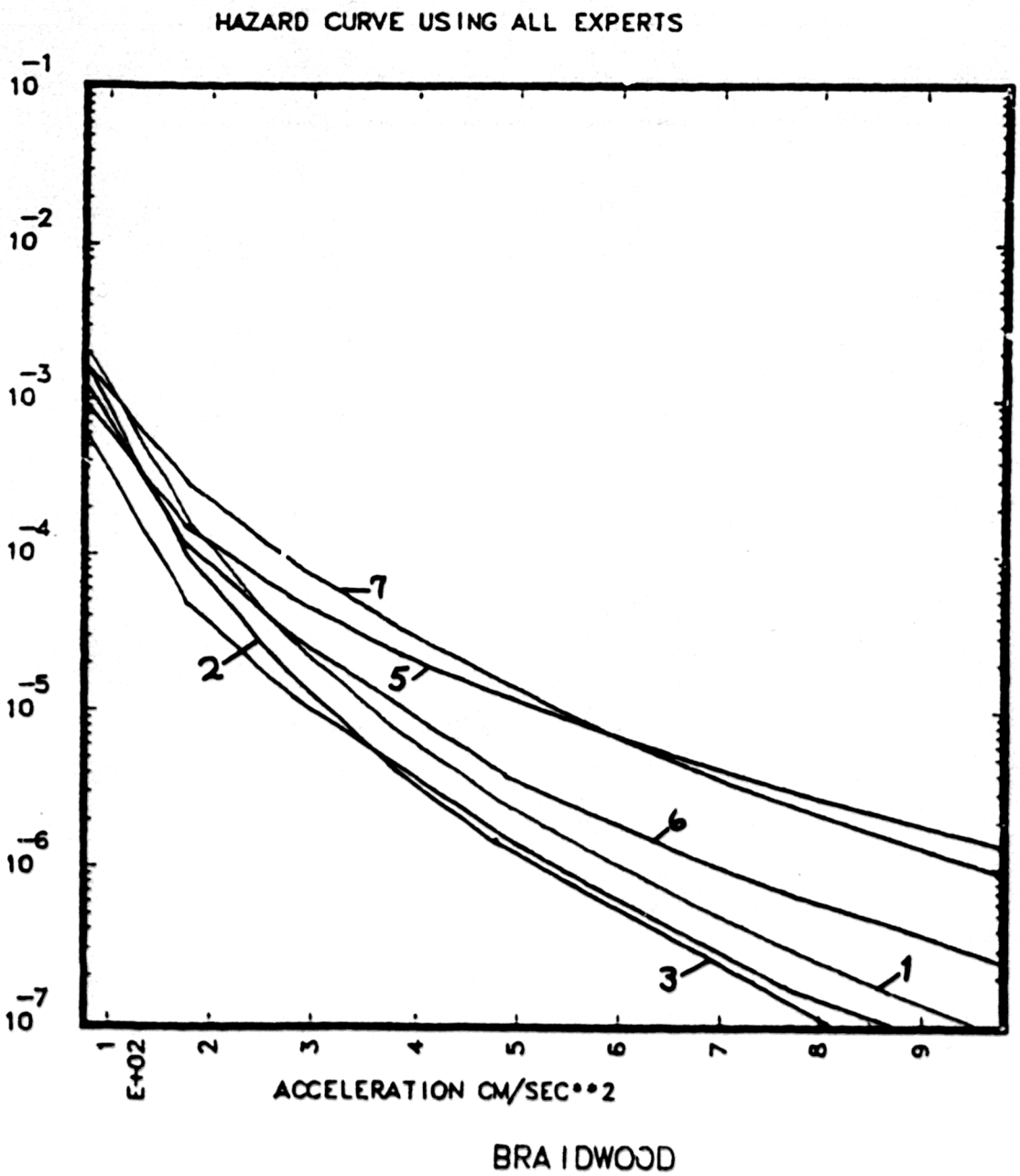


Fig. 3.9 Hazard Combined Over Seismicity Experts 1,6 and 12, Calculated For Each One Of The Ground Motion Models Selected By Ground Motion Expert 1 (Models 1, 2, 3, 5, 6 And 7).

4453

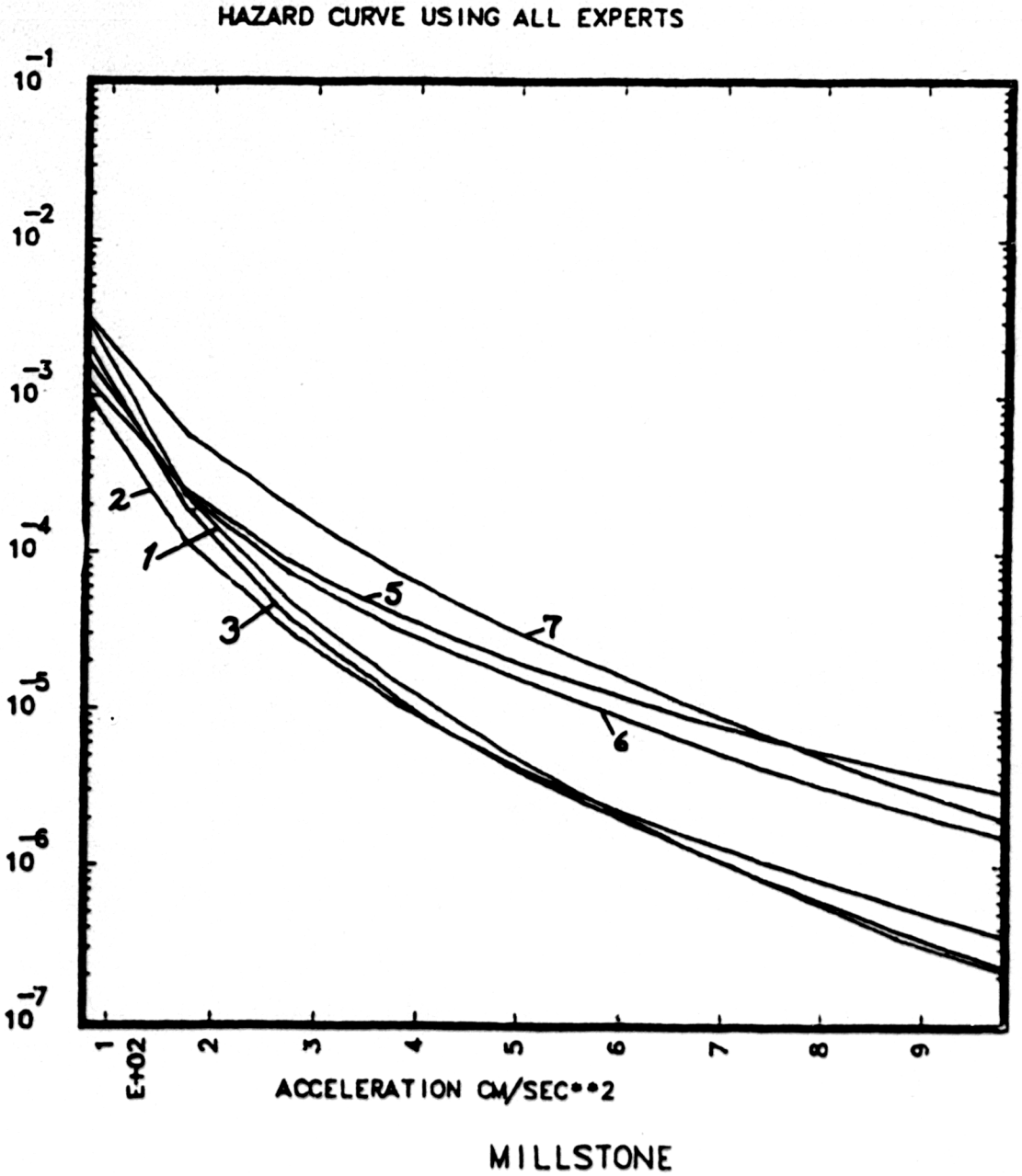


Fig. 3.10 Hazard Combined Over Seismicity Experts 1,6 and 12, Calculated For Each One Of The Ground Motion Models Selected By Ground Motion Expert 1 (Models 1, 2, 3, 5, 6 And 7).

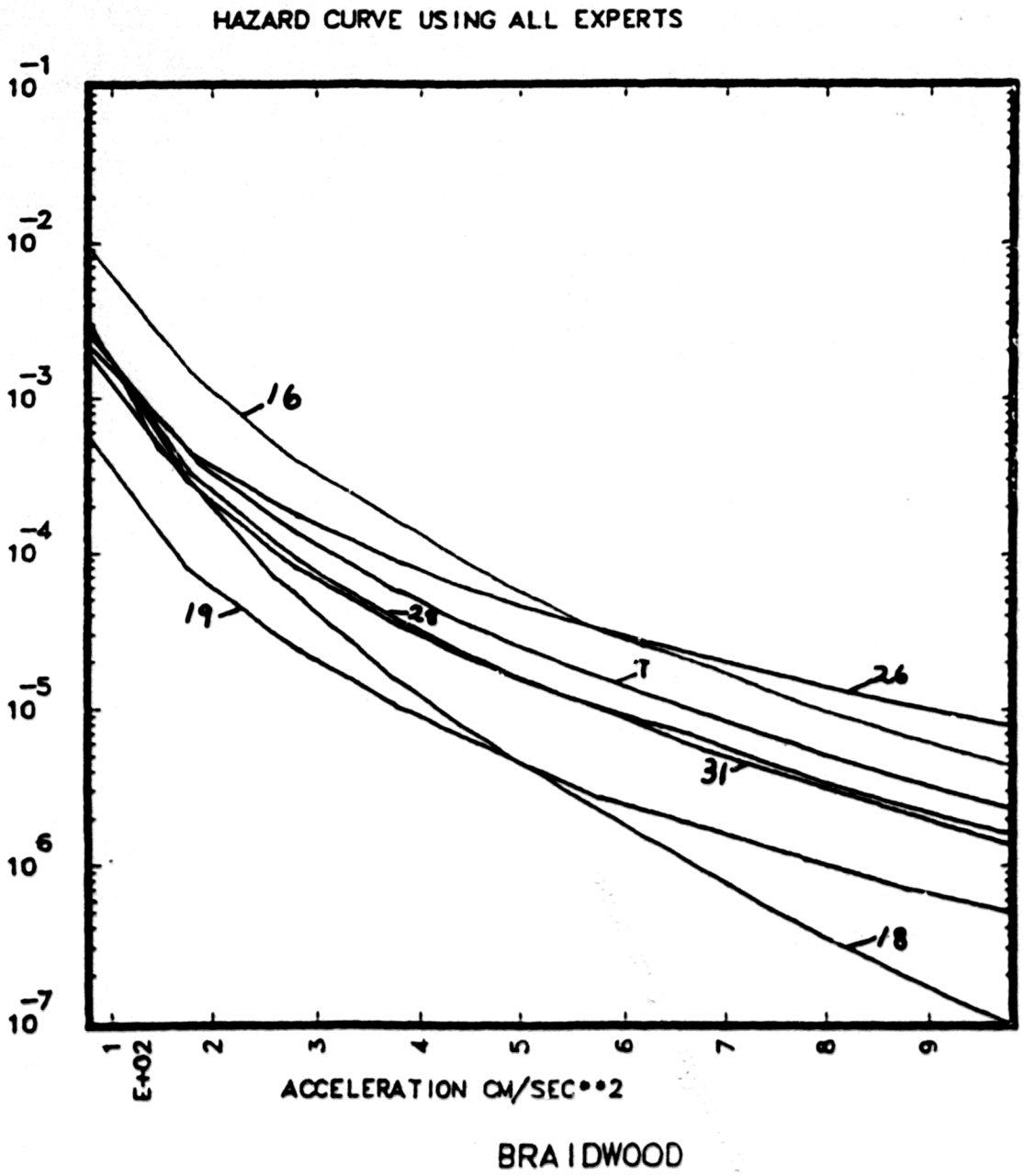


Fig. 3.11 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 2 (Models 7, 16, 18, 19, 26, 28 And 31).

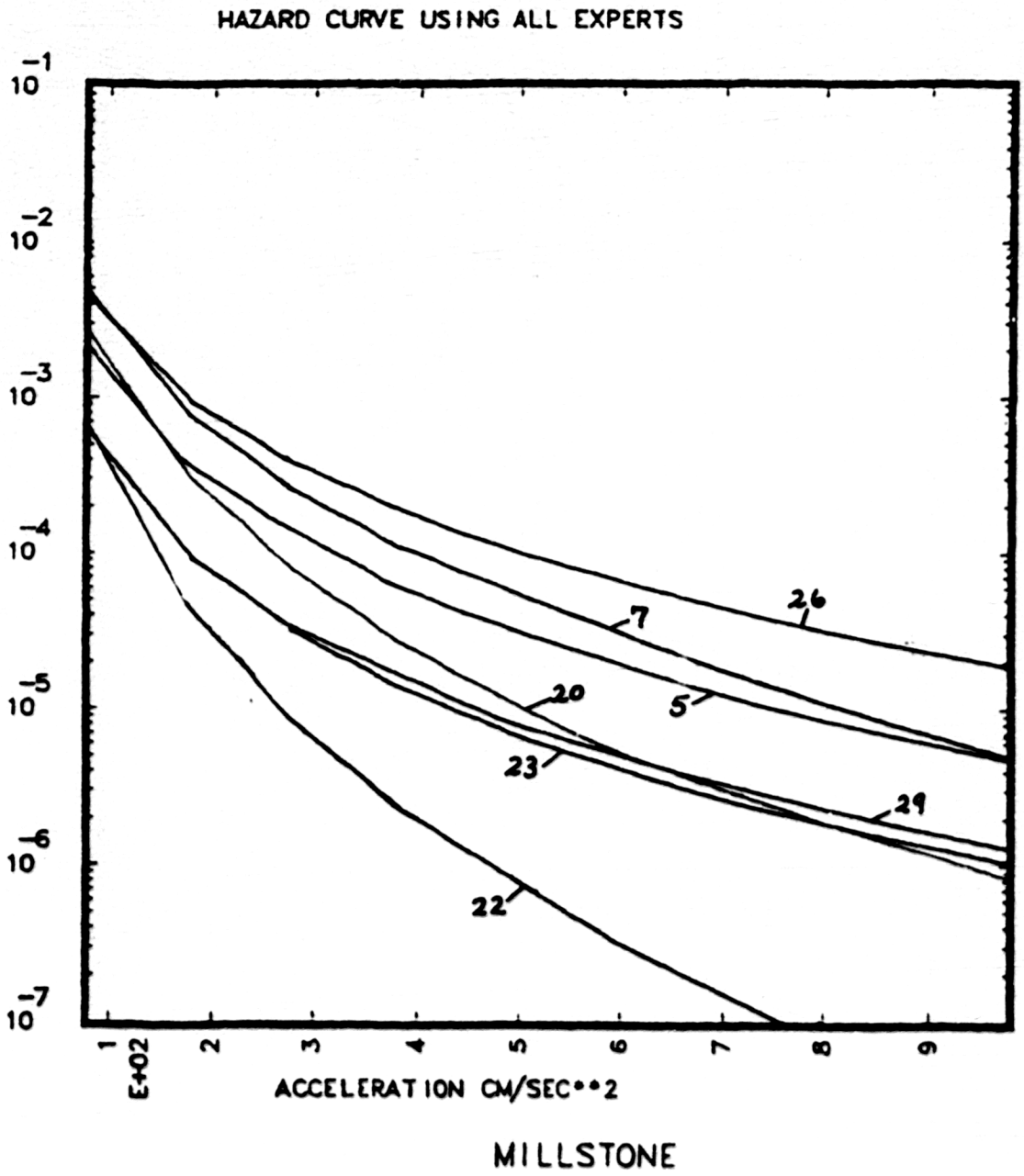


Fig. 3.12 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 2 (Models 5, 7, 20, 22, 23, 26 And 29).

U2159

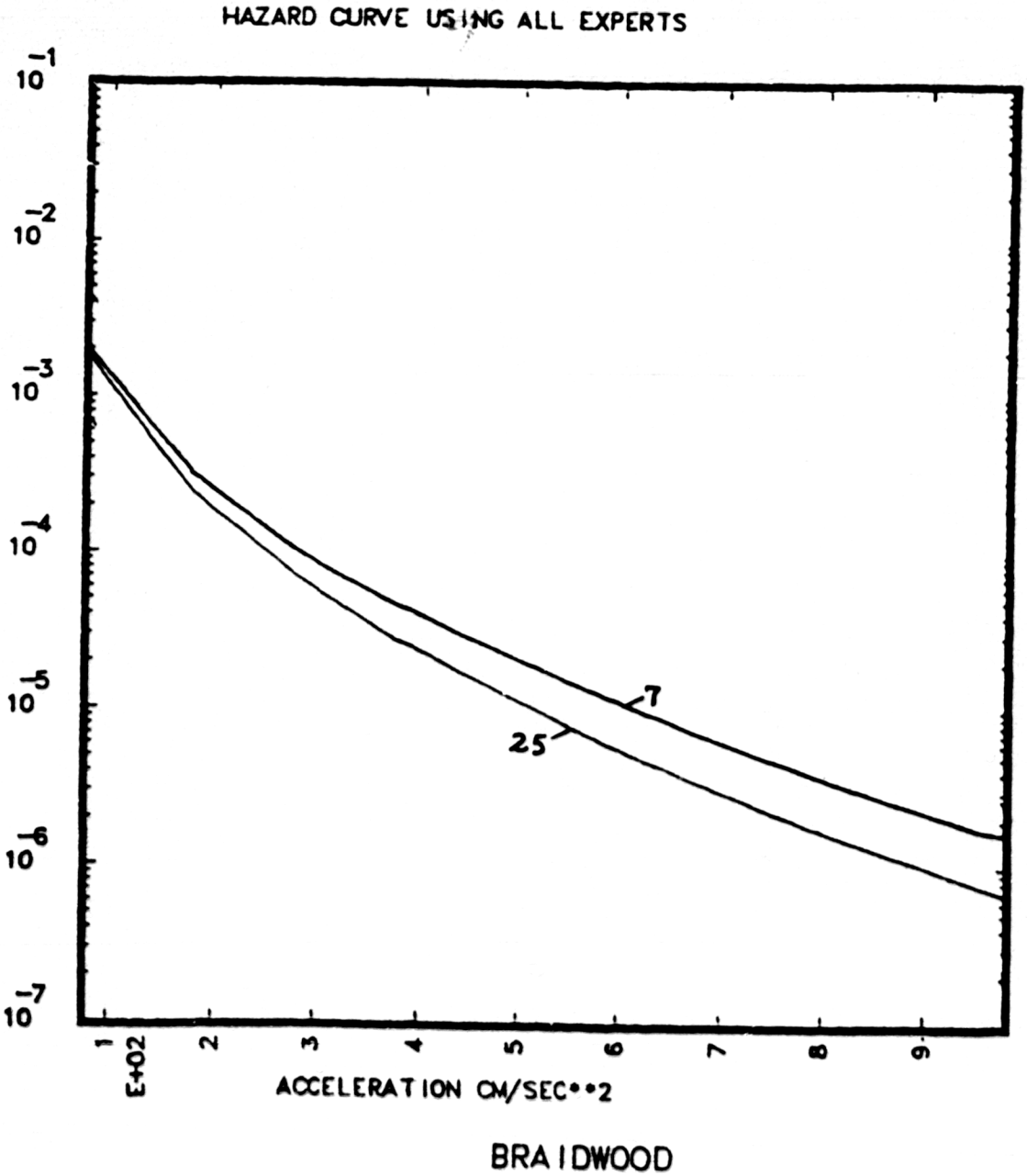


Fig. 3.13 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 3 (Models 7 And 25).

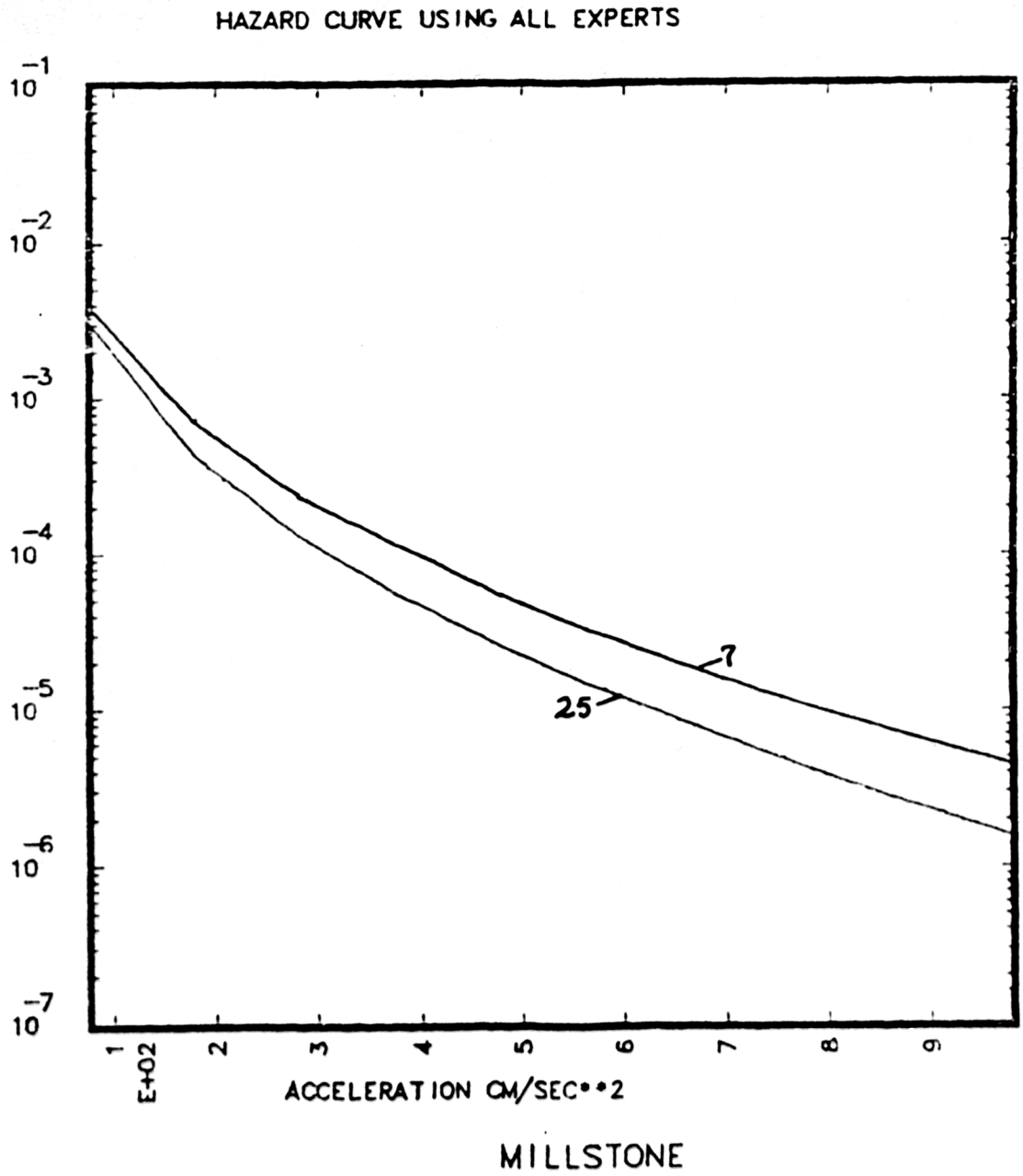


Fig. 3.14 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 3 (Models 7 And 25).

U3059

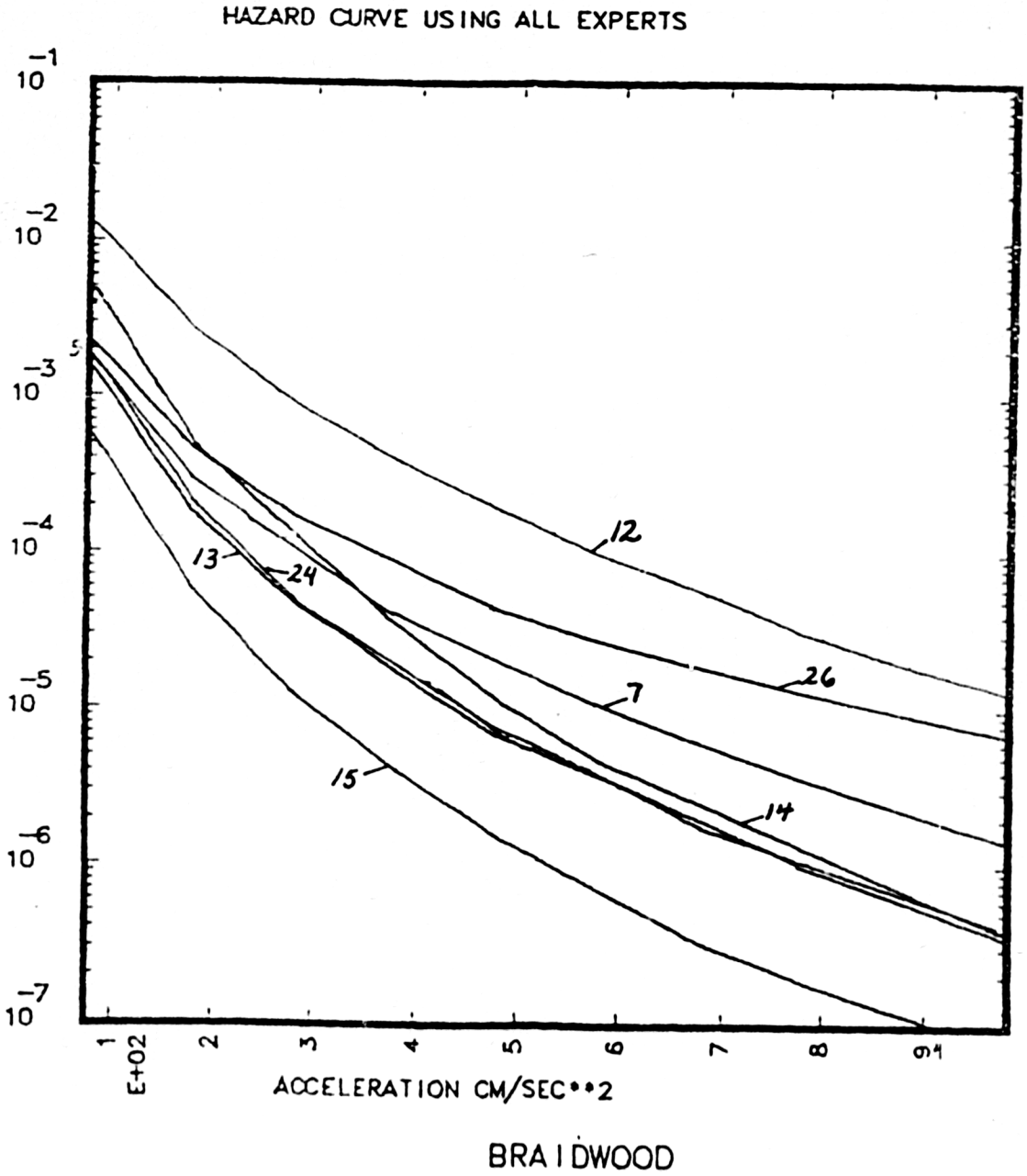


Fig. 3.15 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 4 (Models 7, 12, 13, 14, 15, 24 And 26).

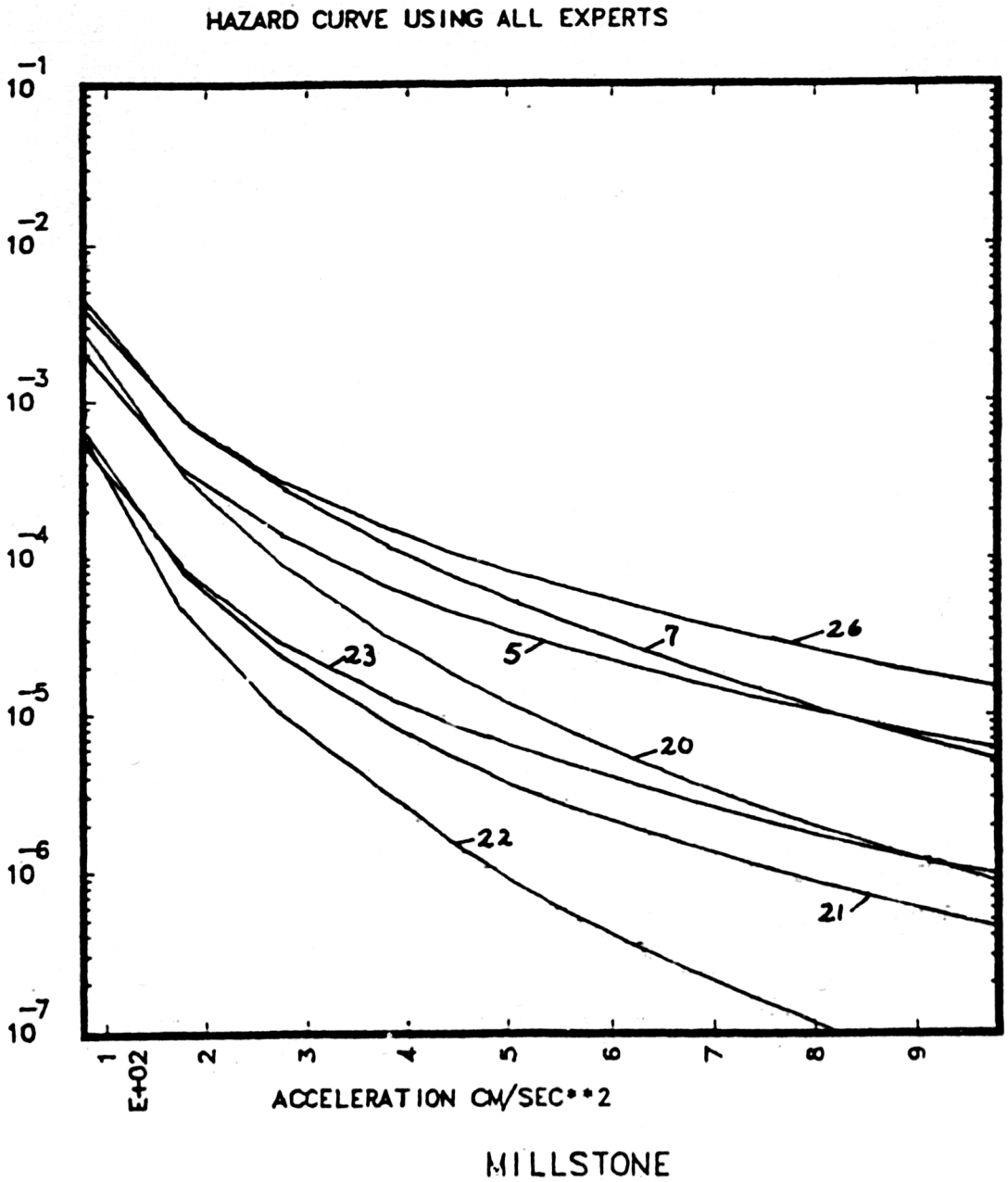


Fig. 3.16 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 4 (Models 5, 7, 20, 21, 22, 23, And 26).

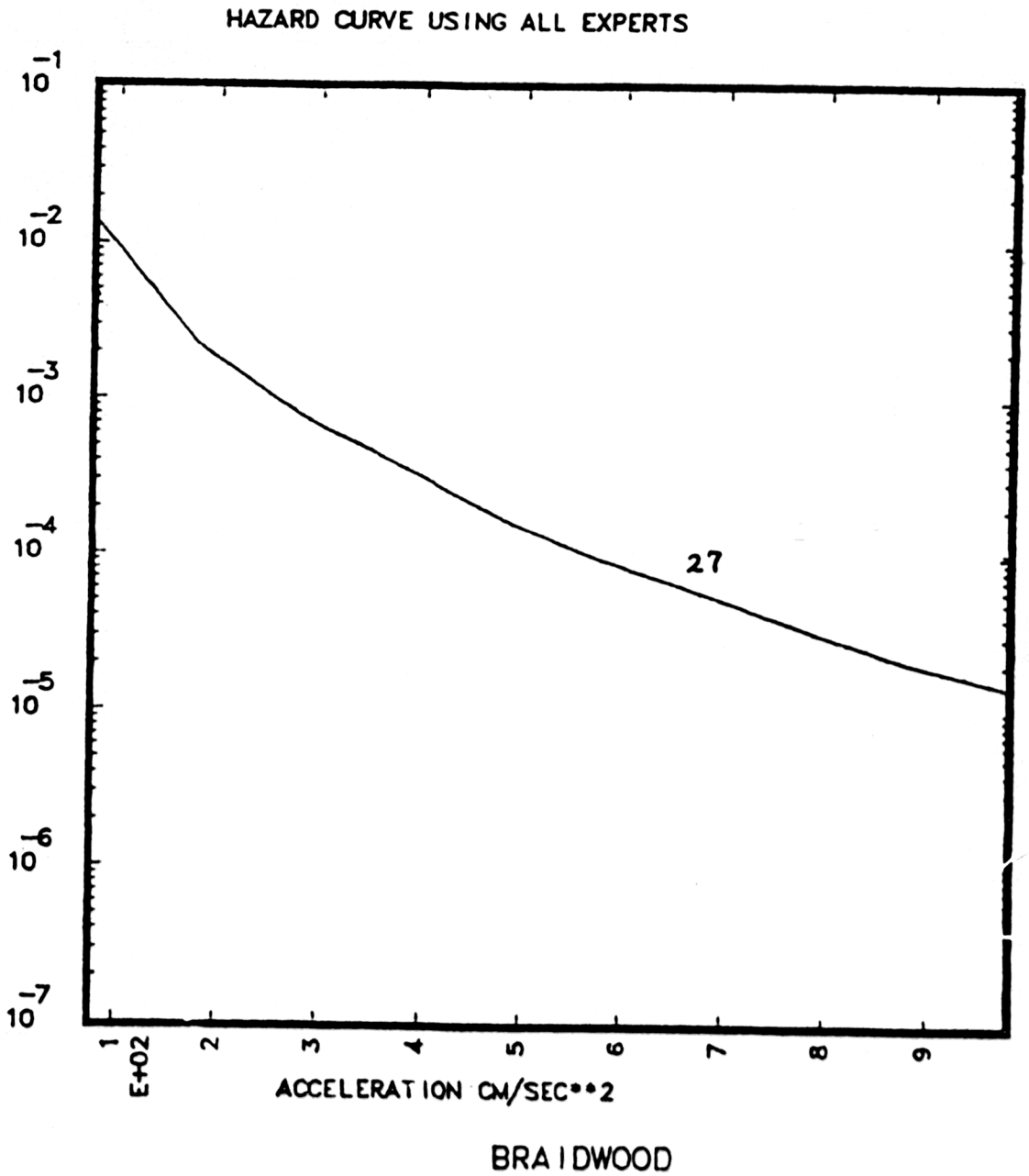


Fig. 3.17 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 5 (Model 27).

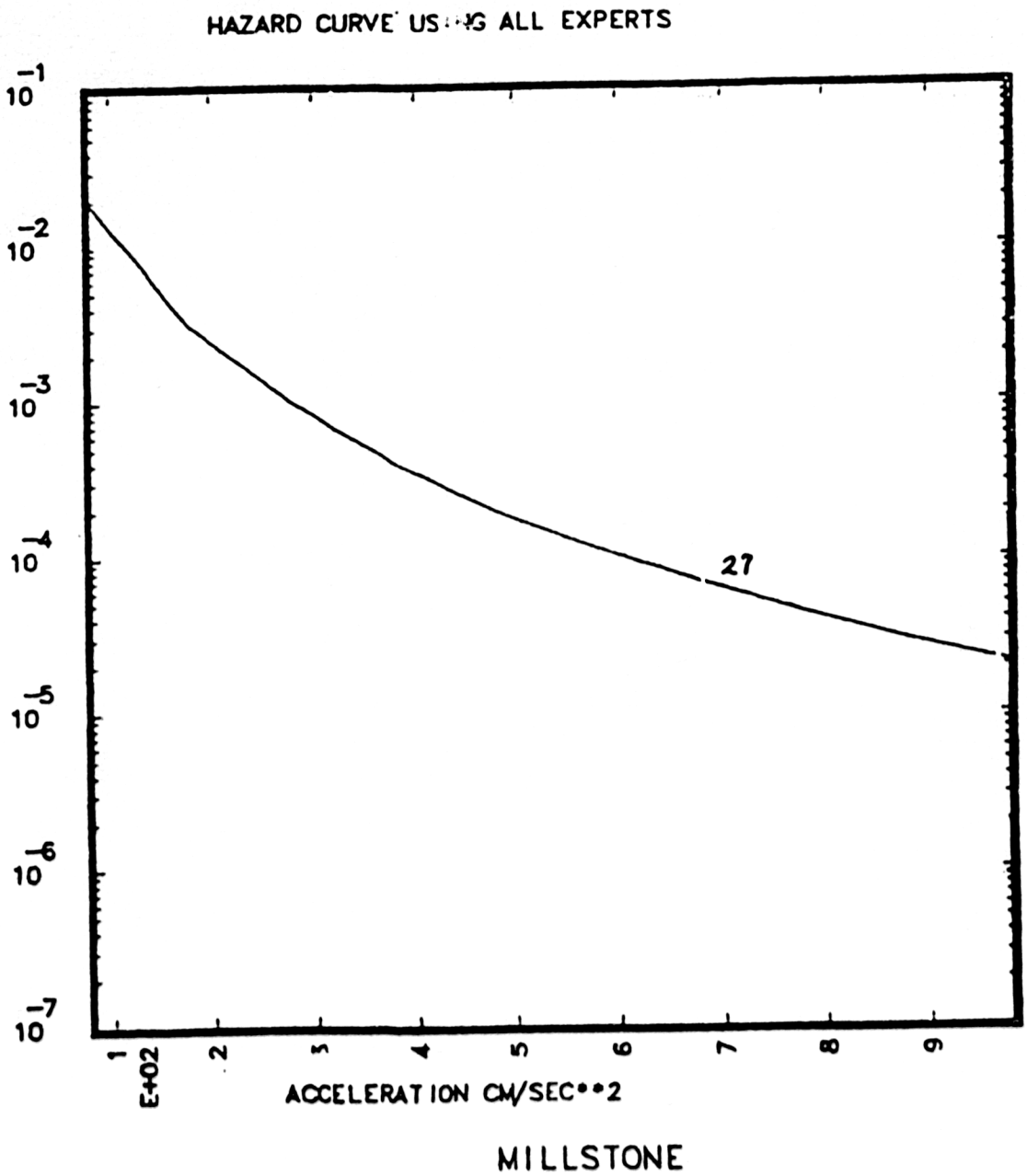
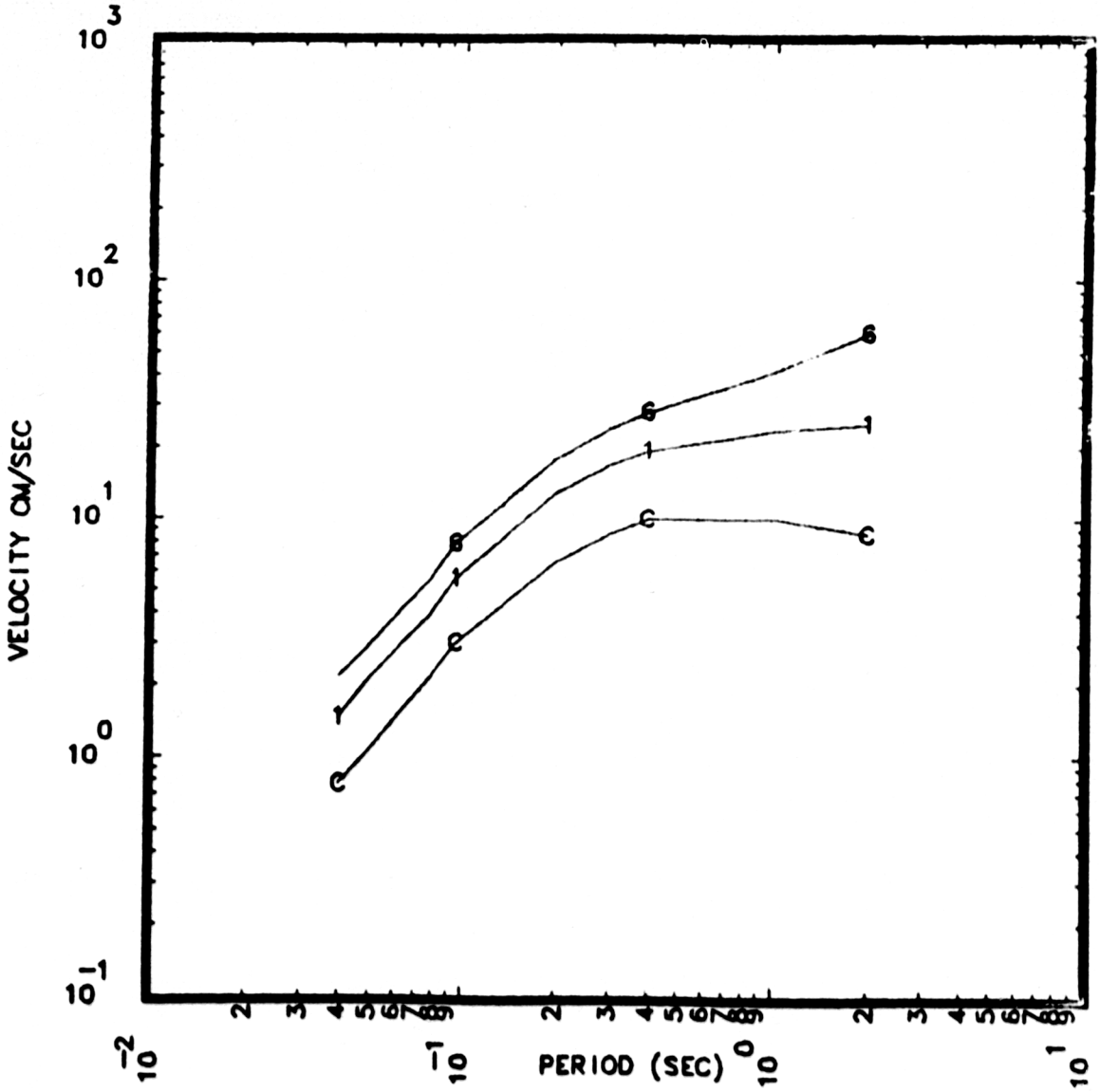


Fig. 3.18 Hazard Combined Over Seismicity Experts 1, 6, And 7, Calculated For Each One Of The Ground Motion Models Selected By Expert 5 (Model 27).

BEST ESTIMATE SPECTRA BY SEISMIC EXPERT FOR

1000. YEARS RETURN PERIOD

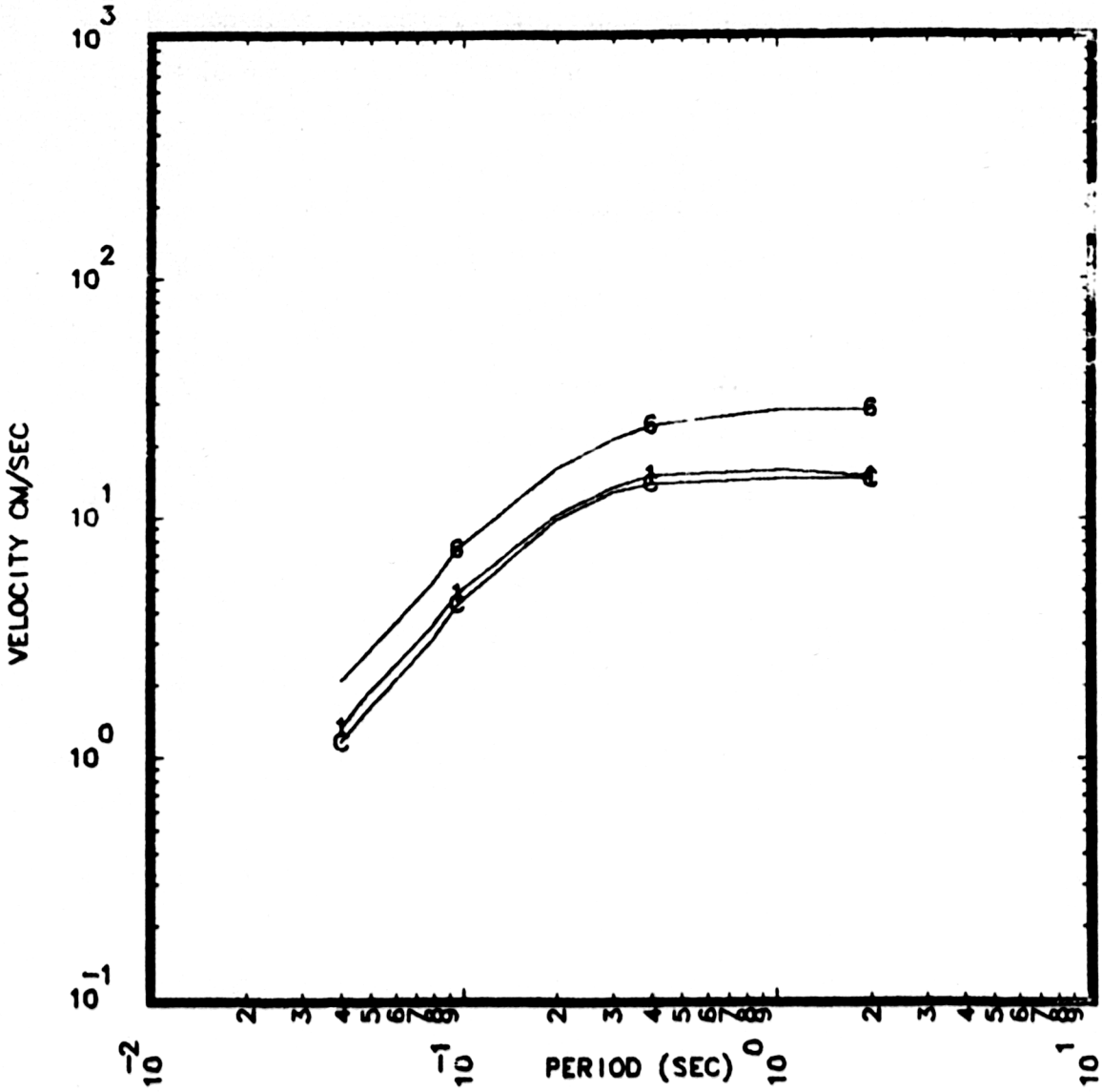


BRAIDWOOD

Fig. 3.19

BEST ESTIMATE SPECTRA BY SEISMIC EXPERT FOR

1000. YEARS RETURN PERIOD



MILLSTONE

Fig. 3.20

4. UPPER LIMIT OF THE GROUND MOTION

4.1 Ground Motion Saturation

It was first suggested by Housner (1965) that accelerations in excess of .5g where not possible to occur, but values much larger than .5g have been recorded since 1965. Later Hanks and Johnson (1976) proposed an upper limit of 1.8g for the acceleration. Even if there is no seismological evidence of higher values by actual recording, there is no statistical evidence either to eliminate the possibility of very high strong motion accelerations.

The problem of identification of the upper limit of the ground motion, which we call here ground motion saturation, might also be different in the Eastern U.S. from what is in the West. Since the EUS earthquakes are believed to be richer in high frequencies than the earthquakes in the West, it is legitimate to consider the possibility of occurrence of peak ground accelerations at higher frequencies than for the West. This also leads us to postulate the possible occurrence of peak ground acceleration in the East higher than anything observed in the West.

In spite of these arguments in favor of high saturation values, common sense and basic physics tell us that any GMP cannot be unbounded.

Bounding the GMP seems a logical step for implementation in a hazard analysis, but then we must ask ourselves the following questions.

1) Does it make any difference in a hazard analysis to consdider the GMP bounded?

2) If the saturation value is very high, compared to the mean, does it make any difference in the hazard computed with unbounded GMP?

3) If the GMP is bounded, what is the saturation value?

The most important question is clearly the first one. Many analysts have tried to answer this question, and they concluded that the saturation value of the GMP is a very important value in assesssing the hazard. However, we have to be careful in reaching a conclusion since the saturation values chosen were all in the range of observed values. For example, in a recent study performed by Zemell (1984), it is shown that imposing a saturation value of 1.8g increases the hazard by 50% from what it would be if the saturation value was 1.2g (i.e: .15g vs .10g) for 10,000 yr return period and only approximately 10% (i.e: .065g vs .058g) for 1,000 yr return period. If we were to perform again those analyses with the same ground motion models and same random variation values but with a higher range of saturation values, we believe that the sensitivity of the results to changes in the saturation would be much smaller than in the example cited above.

This last remark leads us to the next question. Let us assume that the saturation value is very high, and let us model the random variation of the GMP by a truncated distribution (such as the truncated lognormal as in our analysis), truncated at the saturation value. Then the difference with using an umbounded distribution would be small since in the unbounded case the probability of exceeding a value equal to the saturation value, would be very small.

In trying to answer the third question above, we have to keep in mind that we are concerned with a frequency range of engineering interest, i.e., frequencies less than 25Hz. This restriction eliminates the peak ground acceleration that we might observe at very high frequencies. In spite of this restriction of the range of values, evaluating the saturation value still remains a very difficult task with a large uncertainty, as no data exist to help us.

4.2 Modeling the Saturation of the GMP

We mentioned above that any estimate of a saturation value would be very unreliable. Rather than trying to associate some uncertainty with the estimate, we prefer to keep this paramenter as a deterministic parameter as we believe that the task of evaluation of a standard deviation would be too unreliable.

We then propose three simple methods of introducing GMP saturation in our analysis, shown in Fig. 4.1:

- The first method, labeled Type I, assumes that there exists some value a_1 of the GMP which cannot be exceeded at any location. This assumption is guided purely by physical-mechanistic considerations on the resistance of the top layers of soil at site and on the resistance of the rock on the travel path as well as the nature of the earthquak source. This is shown by the Curve C_1 in Fig. 4.1.

- The second method, labeled Type II, assumes that the upper limit of the GMP is a function of the magnitude and distance from the epicenter of the earthquake. It is more directly based on the observed data and assumes the GMP cannot be greater than the average predicted GMP for a given magnitude and distance, multiplied by a constant factor. Thus, for a given magnitude and a given mean curve, the GMP is bounded by the curve C_2 in Fig. 4.1.

- The third method relies on both the assumptions made in the Type I and Type II methods. Thus, for a given magnitude and a given mean curve, the GMP is bounded by the curve c_3 which is the lower envelope of the curves c_1 and c_2 in Fig. 4.1.

In Question 6.2 of this questionnaire, you will be asked your opinion on the necessity of considering that the GMP is bounded. If you are of the opinion that it should be bounded, you will be asked to indicate the model of saturation you consider as the best suited for the analysis, and to provide the needed parameters to define your choice of saturation model.

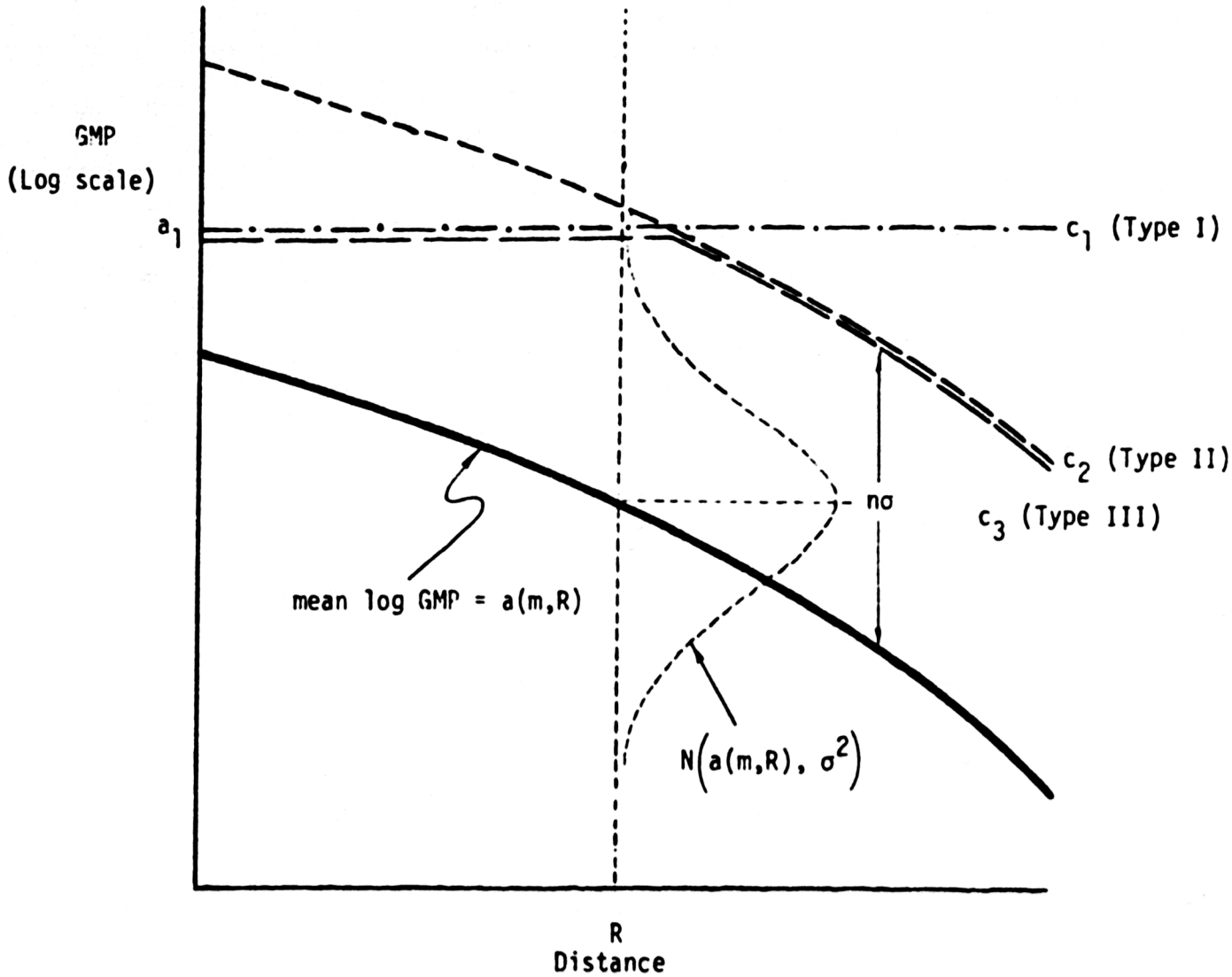


Fig. 4.1 Saturation

Description of the three types of models considered for the physical saturation of the ground motion. The random variation of the logarithm of the GMP is modeled by a normal distribution with mean a(m,R) and standard deviation σ

5. LOCAL SITE EFFECTS

5.1 Background

A possible major deficiency in the hazard curves and uniform hazard spectra presented in the Interim Report lies in the way that the correction for the various sites soil column was incorporated into the analysis. For many ground motion models, e.g., spectral models based on RG 1.60, ATC, or the Newmark-Hall model scaled by Nuttli's velocity model, no correction was entered. For others, only a very simple correction based on use of a simple soil classification procedure such as that introduced by Trifunac and Brady (1975). For example, the spectral model developed as part of the Site Specific Spectra Project (SSSP) was developed using such a simple approach. By this we mean that this ground motion model (the relation between earthquake magnitude, distance and the ground motion at the site of interest) was developed from the set of earthquakes contained in Vol. II of the Cal. Tech. series in the following manner:

1) We separated this data into rock sites and soil sites using our judgment along with reference to the numerous other such division of this data set.

2) Using variations of this data set we developed correlations between the observed ground motion, distance, magnitude and site-type (rock or soil).

Thus, the spectra that we developed are only valid at what we can loosely term average soil or average rock sites.

Trifunac, in his work, used a slightly more complex classification scheme but the basic concept is the same.

The use of such a simple approach could lead to the introduction of significant systematic errors in the analysis. Two major errors can occur:

1) Projected ground motion estimates for a site are based on correlations developed from data obtained at a number of dissimilar sites; hence, there

is little correlation between the spectral peaks from site to site. Thus, the spectral peaks at one site are often averaged with spectral lows for another site. However, a given site may consistently amplify the incoming seismic energy at certain frequency intervals. The net result is that the projected ground motion estimates may be biased on the low side for such a site.

2) The statistic (typically the standard deviation) that measures the dispersion or spread of the data about the mean could be smaller than that for data recorded at different loosely correlated sites.

Our concern about the first possible error is heightened for many EUS sites, because their geological column is significantly different than for the sites making up the WUS. There are typically three major differences between EUS and WUS sites. First, one significant difference is that many EUS sites are underlain by rocks with a higher shear wave velocity (as compared to typical Western U.S. sites). Secondly, many of these sites also share a relatively shallow soil depth varying between 60 and 150 ft. Finally, the soil profile at a number of EUS sites consists of stiff, highly over-consolidated glacial tills which are significantly different than the soil conditions at the sites contained in the WUS data set.

Although our main concern is about site amplification, the value of standard deviation is also of interest because it has a significant effect on the seismic hazard projected for a site.

The ideal way to address these questions would be to develop ground motion models by regression analysis using only data from sites similar to EUS sites. Such an analysis would most likely result in a higher median estimate and possibly in a reduced error term. However, it is not possible to do this because there is so little data from sites with characteristics similar to such EUS sites. A second approach would be to apply correction factors (determined from judgment and analysis such as presented in the following sections) to the uniform hazard spectrum for a given return period.

If correction factors are used, these correction factors (in our opinion) should not be applied to the spectra given in the Interim Report. There are two reasons for this. First, the spectra presented in the Interim Report are the result of combining over a number of different spectral models. These models, for the reasons discussed below in Section 5.3, do not all represent the same base case. The correction factors should be applied in a consistent manner to each ground motion model during the analysis. Secondly, the spectra developed in the Interim Report were computed using an error term for the ground motion model based on data recorded at many different sites. The error is made up of source, travel path, site, and because intensity was used in some cases, building effects. All contribute significantly to the uncertainty. The contribution to the uncertainty due to the data being recorded at different site types and due to role of site intensity should be removed from the estimate of the error term used for the ground motion models in the hazard analysis. The uncertainty in the location of median estimates should be included in the correction factors used.

5.2 The Need for Correction

A reasonable starting point is to address the basic question, is site response an important phenomena that significantly affects the level of strong shaking at ground periods of engineering interest? This question has been frequently adressed in the literature. Most recently, a three day workshop was held at Santa Fe to address this question. One of the papers at this meeting, Rogers et al (1973) noted that the papers at responses to this ranged from "very significant" to "not important in this region" to "important, but impossible to predict." The preponderance of data, however, including damage statistics, intensities, strong motion measurements, and low-level ground motion measurements indicates that site response effects are significant. But there still exists significant controversy as to how important these effects are (given the differences in motions caused by different source mechanisms and different wave-transmission paths) and as to what is the best analytical procedure to evaluate them. The absence of recorded field performance has contributed to the continuing debate on this subject.

It seems that the heart of the problem lies in the difficulty to accurately predict the wave content of a potential earthquake motion. The available procedures are usually based on the assumption of vertically propagating shear (S) waves (1-Dimensional Theory). Many experts in earthquake engineering, however, are unwilling to accept the results of 1-D amplification because, they feel, this theory predicts a greater degree of amplification over a narrow frequency range than is actually observed in nature, except in unusual circumstances. The lack of consipicuous soil effects in the San Fernando 1971 earthquake records has given some support to these arguments, e.g., Crouse (1978). It has also been argued that if soil amplification theories were true, all earthquakes recorded at a site would have very similar frequency content.

On the other hand, the Japanese and Russians have examined numerous case studies showing that the geographic distribution of damage and intensity is in some way related to the thickness and/or other physical properties of the near-surface sediments (Kanai, 1952, Ooba, 1957; Minakami and Sakuma, 1948; Medvedev, 1962). Using recordings of low-level ground motions produced by distant nuclear explosions, Borcherdt and Gibbs (1976) found a high correlation between geographic changes in intensities in the 1906 San Francisco earthquake and geographic changes in mean spectral amplification. This correlation exists up to the highest shaking levels that occurred in that earthquake (Modified Mercalli $\geq$ X). Site effects have recently been isolated in the strong motion data from the 1979 Imperial Valley earthquake (Mueller and others, 1982) and are also observed in the strong motion data recorded during the Fruili and Ancona, Italy earthquakes (Chen et al -- Chiaruttini and Siro, 1981). Analysis of the San Fernando earthquake data set by Rogers and others (1983a) shows that site effects occur at the 28 strong-motion sites they studied. For these sites mean spectral amplification on soil ranged as high as a factor of 5. Data at other Los Angeles sites, employing recordings of Nevada Test Site nuclear explosions, indicated that at some alluvium sites mean spectral amplifications as large as 11 are observed. Many other examples of ground motion studies could be cited that demonstrate the importance of geologic conditions on strong ground shaking.

Appendix B of this report contains a discussion of the reasons why we consider it necessary to include a correction for the local soil column at various sites.

5.3 Proposed Approach

5.3.1 Overview

One of the main objectives of this project is to assess the uncertainty in the estimate of the seismic hazard at selected nuclear power plant sites in the EUS. In keeping with this objective, we want to include the uncertainty introduced by the local site condition at various power plant sites. This uncertainty has both random and systematic components. The systematic component can be accounted for by using several different approaches to obtain the correction factors varying from "no correction" to that obtained by a linear 1-D analysis such as performed in the SHAKE computer program. The random aspect which arises from our uncertainties in the soil column and energy and frequency content of the potential seismic ground motion at the site, can be accounted for by including uncertainty in the correction factors for each systematically different method used to develop the correction factors. Your role as Panel Members will be to select the approaches/correction values you deem best and give subjective weights, i.e., the likelihood that they are correct relative to the other approaches/correction factors.

There are a few important limitations to what is possible, e.g. schedule and budget requirements currently preclude the development of some new approaches. Schedule and budget requirements also restrict what we can do, i.e., acceptable approaches must fit into our analysis scheme. Because of these limitations, our questions in Section 6 are structured into two parts. The first part deals with a limited subset of approaches/correction factors which we can include in an analysis under current schedule and budget limitations. We also solicit your ideas along these lines. We anticipate that at a number of sites the simple approaches/correction factors will have a significant effect on the estimates of the seismic hazard at those sites. Thus, for the longer term we will also suggest/solicit more complex but presumably better ways to incorporate the significance of local site effects into the analysis for NRC's future consideration.

For the short term we propose the following approaches/corrections factors:

1) No correction.

2) Use only a simple soil or rock classification if available -- otherwise, no correction.

3) Develop correction factors for each ground motion model based on several generic site classifications, 1-D analysis data, and judgment.

4) Do a site specific analysis.

5) Other - as proposed by panel members.

Each of these are discussed in detail in the following sections:

5.3.2 No Correction

Here, the argument might be that both our knowledge of EUS ground motion is so poor and the methods we have to assess local site effects so uncertain, that it would be better to do nothing. All site types would be treated the same. Here, it should be noted that some of the ground motion models adopted seem to fall into this category. For example, Nuttli's model #7 makes no reference to site type. One could argue that it is for "generic" soil sties. Certainly, Campbell's models fall into the "generic soil" category as they were developed using only soil data. For other models, e.g. what we have labeled the Trifunac-Anderson model, have a simple site correction term included. If this case is selected and such a ground motion is included, then the value for the site correction term should be specified.

5.3.3 Simple Rock/Soil Correction

For this case, the site types are put into two (rock or soil) or (stiff or soft) or three categories (soft, stiff, basement rock) and a simple constant (for each category) correction factor is applied.

Figure 5.3.1a shows typical correction factors going from soil to rock as a function of period found from WUS data. The curve labeled 1 is based on Joyner-Boore (1982) regression analysis, the curves labeled 2 and 3 are based on Trifunac and Anderson (1977) and the curve labeled 4 is based on the SEP results Bernreuter (1981). For Trifunac's model, curve 2 is between soft alluvium and hard sedimentary rock and curve 3 is between soft alluvium and basement or crystalline rock. Both Joyner-Boore and the SEP use only rock/soil categories.

Figure 5.3.1b is similar to Figure 5.3.1a but now the correction is from rock to soil.

The ratios between Rock/Soil for PGA and PGV are given in Table 5.3.1.

TABLE 5.3.1

Model	Ratio PGA	Ratio PGV
Trifunac (1976a) Basement Rock	1.93	1.07
(Intensity Based) Sedimentary Rock	1.4	1.03
Joyner-Boore (1982)	1.0	0.68
SEP	1.0	0.87
McGuire (1978)	1.22	0.93
Trifunac (1976) Crystalline Rock	0.76	0.55
Sedimentary Rock	0.87	0.74
Campbell (1981)	1.0	---

It can be seen from Table 5.3.1 that there is considerable variation between different studies. These differences arise from several different causes. The first is that form of the model used may not be applicable, i.e., the influence of site type might be a function of magnitude and distance. This is very hard to verify because there are too few sites which are truly rock sites. McGuire (1977) developed a relation between site intensity and the peak ground motion parameters:

$$\ln(a) = \begin{cases} -0.83 + 0.85\, I_s & \text{(medium sites)} \\ 0.27 + 0.6\, I_s & \text{(soft sites)} \end{cases}$$

$$\ln(v) = \begin{cases} -4.02 + 0.95\, I_s & \text{(medium sites)} \\ -1.51 + 0.54\, I_r & \text{(soft sites)} \end{cases}$$

whereas Trifunac (1976) fit the relation

$$\log a = C_1 + C_2\, I_s + C_3 S$$

where S is the site type variable

S = 0 soft
= 1 sedimentary rocks/stiff
= 2 basement rocks

In the same 1977 study, McGuire also found

$$\ln a = \begin{cases} 1.81 + 0.9\, M_L - 0.9 \ln r & \text{(soft)} \\ 1.47 + 1.0\, M_L - 0.9 \ln r & \text{(stiff)} \end{cases}$$

$$\ln v = \begin{cases} -1.58 + 1.0\, M_L - 0.7 \ln r & \text{(soft)} \\ -3.61 + 1.4\, M_L - 0.8 \ln r & \text{(stiff)} \end{cases}$$

These results would suggest that the use of a simple distance - magnitude independent correction factor might be too simple. It should be noted that in his 1978 paper, McGuire used a simple constant correction factor to account for site conditions in his regression analysis model. He indicates that he did investigate a case where he modeled the site conditions with a relation of the form

$$(C_M M + C_R \ln r)^S$$

where s = 0 for rock sites
= 1 for soil sites

McGuire concluded that the coefficient C_R was not statistically significant, however, the coefficient C_M was. He noted that the use of a more complex model did not improve the fit to the data, therefore, there was no point in using the more complex model. Unfortunately, McGuire did not report the value of the coefficients C_M and C_R. In addition to possible deficiencies in the form of the mathematical model used for the regression analyses, all of the regression analyses were performed using less than perfect data sets. All the data sets suffer from the use of a poor criteria for the identification of rock sites. Recently, more site boring data has become available to assist in properly sorting the data into categories. The Joyner-Boore (1981) data set is the best in this regard, but it contains a number of questionable sites identified as rock sites. Except for the Joyner-Boore data set records recorded in large buildings and/or in basements, were included in the other data sets used to obtain the results plotted on Figure 5.3.1 and given in Table 5.3.1. Joyner and Boore (1981), Campbell (1981) and (1983) and others have shown that building type and location of the recorder in a sub-basement can have a significant effect on both the PGA and PGV.

The major problem with using the simple category approach is that ground motion models that incorporate such corrections were generally lowly weighted. Only one expert chose such a model as his best estimate model - Model #27 (PGA), #125 (S_V). The other best estimate models, e.g. Nuttli's Model #7 (PGA), #37(V) are based on all data (rocks and soil) or just limited to soil. However, there are so many more data points on soil than rock that effectively the models are "soil" models.

One possible approach would be to introduce a correction factor for these models based on WUS data, i.e., use one of the factors plotted on Figure 5.3.1 to convert from "soil" to rock. This is a somewhat arbitrary approach but would include some correction for the systematic difference that exists between sites. PGA could be corrected using one of the factors from Table 5.3.1. In a somewhat more complex model median, correction factors could be given along with the uncertainty (and a model for the distribution) of the median.

5.3.4 Generic Correction Factors

5.3.4.1 Overview of Approach. The simple model proposed in Section 5.3.3 might be adequate if enough categories are used; however, the data base is too sparse to define many categories. One possible approach, developed in this section, is to supplement the data set with analysis. Our proposed procedure is as follows:

1) Use available soil/rock pairs (soil and rock stations in close proximity to each other that record the same earthquakes) to compute empirically observed amplification factors. This provides a measure of the range of amplification observed and the uncertainty introduced by source, travel path and rheological effects. These results are used to calibrate analytic results.

2) Ideally, one would like to have a sufficient number of soil/rock pairs to put them into a "reasonable" number of categories based on soil type, depth, bedrock shear wave velocity, etc. and develop "generic" median amplification factors (as a function of at least frequency) for each category and the uncertainty associated with each category. Because sufficient data does not exist to even make a "rough cut" at this time, we must resort to analytic modeling. We defined eight categories based on three soil depth categories and two soil type categories plus a rock category and a deep soil category. Two basic soil types were chosen: (1) primarily a sandy type soil column and (2) primarily a "till like" column. Granted, most soil columns are mixed, but defining too many categories becomes pointless and a site specific approach such as discussed in 5.3.5 should be used. Each category contains several different soil columns which are based on actual soil columns at nuclear power plant sites. To incorporate the uncertainty from the source and travel path effects in the analysis, we selected a set of time histories recorded at rock sites with a range of magnitude and distances. These time histories were used as input to the SHAKE computer program and the PGA and amplification factors were computed for each category for each time history. Then for each category, a median amplification factor (correction) and its uncertainty was computed.

One major question arises -- what should these correction factors be applied to? Ideally, we should predict the PGA hazard curve and UHS at each site at a hypothetical unweathered basement rock outcrop and then apply the correction for the appropriate site category to obtain the PGA hazard curve and UHS corrected for local site effects. Unfortunately, it is our judgment that none of the ground motion models available can be considered as predicting the ground motion at an unweathered hard rock outcrop. In fact, the models most highly weighted by the Ground Motion Panel can at best be considered as applicable to only a "generic" soil site. A few of the models chosen can be considered applicable to weathered rock sites. But in our opinion, these models have the serious problem that the different data sets used to develop them have shallow soil sites listed as rock sites and they have data from large buildings/basements intermixed with free-field field data. Also, the set of time histories used to develop the correction factors were generally recorded on highly weathered rock.

Given the lack of data recorded at true rock sites and the possible complexity of the systematic differences between rock and soil sites, it is not clear that even with added analysis that we could develop an acceptable ground motion model for even weathered rock sites. For this reason we included a "generic" soil category in our analyses. Amplification factors were computed relative to the generic soil category as well as rock. Several avenues are possible: (i) Only use the soil version of the various ground motion models and correct using computed correction factors for the site's category. (ii) Use a mixed set; i.e., use the rock version, if available, and use the rock to site category correction factor; otherwise use the soil model and soil to site category correction factor. (iii) Convert all models to a rock version using the approach suggested in Section 5.3.3 and then use the rock to site category correction factor.

5.3.4.2 Selection of Time Histories. Ideally, we would like to have a set of time histories recorded on unweathered hard rock from earthquakes with magnitudes ranging from 4.0 to 7.0 and distances ranging from 2-1/2 km to several hundred kilometers. With such a set, we could rerun our analysis and then examine the results to see if magnitude and distance were important.

Unfortunately, the available set of time histories does not match the ideal set. Most records are recorded on weathered rock -- and in fact, the shear wave velocity of the "rocks" at many sites is closer to a "soil" than a rock. Because only ratios are involved, this may not be a major problem except at hard unweathered rock sites where we do not have a good measure of what the difference might be between the ground motion recorded at hard rock sites as compared to soft weathered rock sites.

Table 5.3.2 gives a list of the records selected. Because of time and budget constraints we have limited our analysis to twenty records using only the maximum horizontal component from each station. We restricted our choice to recordings made either in the free-field or in small buildings. It can be seen from Table 5.3.2 that we have a reasonable distribution of magnitudes but not a very good distribution on distance.

Table 5.3.2

Rock Records Used in the Analysis

Station	Earthquake	Mag. M_L	Dist. R	Accel. g's
Helena Fed. Bld.	Helena, Mont. 10/31/35	6.[illegible]	8	0.15
Golden Gate Park	Daly City 3/22/57	5.3	8	0.13
Temblor	Parkfield 6/21/67	5.5	11	0.41
Pacolima Dam	San Fernando 2/9/72	6.4	3	1.20
Pacolima Dam	After Shock	5.4	12	0.11
Cal. Tech. Seism. Lab	San Fernando 2/9/72	6.4	18	0.19
Griffith Park Obs.	San Fernando 2/9/72	6.4	17	0.18
Cape Mendocino	Cape Mendocino 6/7/75	5.3	25	0.20
Oroville Seism. Sta.	Oroville 8/1/75	5.7	8	0.11
Gilroy Array No. 1	Coyote Lake 8/6/79	5.9	9	0.13
Gilroy Array No. 6	Coyote Lake 8/6/79	5.9	4	0.42
Superstition Mt.	Imperial Valley 10/15/79	6.6	25	0.21
Cerro Prieto	Imperial Valley 10/15/79	6.6	24	0.17
Superstition Mt.	Westmoreland 4/26/81	5.7	13	0.11
Rocca	Ancona, Italy 6/14/72	4.7	6	0.55
Rocca	Ancona, Italy 6/14/72	4.2	6	0.45
San Rocco	Friuli, Italy 9/15/76	6.1	9	0.12
San Rocco	Friuli, Italy 9/15/76	6.0	19	0.23
Bagnoli	Campunia Lucania 11/23/80	6.7	12	0.18
Sturno	Campania Lucania 11/23/80	6.7	18	0.23

5.3.4.3 Definition of Site Categories

In order to define site categories, site data for more than 60 nuclear power plant sites throughout the United States has been reviewed. Such site data includes geologic profile (layering and depth to rock), soil parameters (soil type, shear wave velocity, compressional wave velocity, density, shear modules and damping ratio at high strain levels) and bedrock properties (shear, compression wave velocity and density). Like most site classification systems, soil depth to the bedrock and soil type are the primary site parameters used to define site categories. Based on our review of the available data from FSAR and PSAR of U.S. nuclear plants sites we have defined the following categories based on the range of the thickness of the soils above bedrock and primary soil type:

Site Class I: Rock Sites. This category includes sites with exposed bedrock including plutonic, igneous, metamorphic, crystalline and sedimentary rock. Sites where the thickness of the soil is less than 25 feet are also assumed to fall in this category and the surface material is neglected as it is generally removed. The mean shear wave velocity from 60 sites is about 6200 fps with a coefficient of variation (COV) of 40%.

Site Class II: Intermediate thickness soil sites. This category includes sites having soil layering thickness ranges from 25 to 300 feet over bedrock. Based on the samples distribution of available sites, we further classify this site class into three subclasses IIa, IIb and IIc.

(IIa): Soil deposit of 25 to 80 feet over rock as shown in Fig. 5.3.2a. The mean shear wave velocity for each site is calculated by weighted sublayer thickness of the site. Among the 12 sites, the mean shear wave velocity is 1500 fps with a COV of 40%. The mean thickness is 48 feet with a COV of 30%. The mean and COV of bedrock shear wave velocity are 6000 fps and 30% respectively.

(IIb): Soil thickness of 80 to 180 feet over rock as shown in Fig. 5.3.2b. The mean and the COV of the shear wave velocity of the 12 sites are 1550 fps and 40% respectively. The mean soil thickness is about 120 feet with a COV of 25%. The mean shear wave velocity of the bedrock is 6400 fps with a COV of 40%.

(IIc): For soil depth of 180 to 300 feet over rock as shown in Fig. 5.3.2c. Only four sites fell into this category. The mean shear wave velocities of soil and rock are 2000 fps and 9350 fps respectively. The mean soil thickness is 250 ft. No COVs were computed due to insufficient site data available to us this time.

Site Class III: Thick soil sites. This is our generic soil category site and includes those sites having soil deposit more than 300 feet over the bedrock. Fig. 5.3.2d shows the shear wave velocity profile for this site class. The mean and COVs of the shear wave velocity among a set of 14 sites are 2115 and 26% respectively. The median value of soil thickness was found 650 feet with a COV of 40%. The mean shear wave velocity is 5700 fps with a COV of 45%.

As these site profiles show a great deal of variability in site parameters, a generic site class can only be defined in a loose manner. For each site class, we only consider two extreme soil types, namely till-like soil and cohesionless soil (sand-like). Since the variation of shear wave velocity with depth for a till-like soil is not significant, a mean constant shear wave velocity was assumed in each site class for the generalized till-like site model for response calculation. However, because the shear modulus for cohesionless soils is more sensitive to the soil depth, it is assumed that the shear modulus varies with the square root of the effective over burden pressure for the sand-like site models.

Table 5.3.3 The means and COVs of the input parameters used for numerical simulation for Site Response Analysis

	Class IIa		Class IIb		Class III	
	Mean	COV	Mean	COV	Mean	COV
H (ft; soil)	48.	0.25	120.	0.25	650.	0.25
Vs (fps; soil)	1500.	0.40	1550.	0.40	2115.	0.40
D (%; soil)	7.	0.60	7.	0.60	7.	0.60
Vs (fps; rock)	6200.	0.40	6200.	0.40	6200.	0.40