Investigation of the July 31, 1997 Collapse of the Parking Garage Steel Framing at the Portland International Airport Portland, Oregon

Office of Engineering
Directorate of Construction
Occupational Safety and Health Administration
US Department of Labor
Washington, D.C.

January 1998

This report was written by
Mohammad Ayub, PE
Fragrance H. Liu, PE

Contributions to this report were made by
Dave Wooley, Chief Investigator, Oregon OSHA,
Rabi Tamerhoulet, PE, Oregon Building Codes Division, and
Dale Cavanaugh, PE, Federal OSHA, Region X.

Figures 2 thru 6, 13 thru 18 were prepared by Rabi Tamerhoulet, PE

Description of the Project and the Incident:

On July 31, 1997 at about 3:30 P.M., two partially completed bays of structural steel, most of which were erected the same day, collapsed killing three iron workers at the southeast side of the new vertical addition to the existing parking garage at the Portland International Airport in Portland, Oregon. The deceased iron workers were on the top of steel girders waiting for one of the braces to arrive which was only moments away.

The existing two-story precast concrete parking garage had three levels of parking. Construction was underway to add four additional levels of parking to transform the garage into a seven level parking structure. The new structure, above the existing structure, was framed in structural steel with a typical bay of 30' × 60'. The floor to floor height was about 11'-6"±. The typical composite construction consisted of girders running in the shorter direction and filler beams in the longer direction. The parking deck floors, to be completed later, comprised of 3" metal composite deck and 3½" concrete with shear studs welded to the beams and girders. The lateral load resisting system was designed to consist of rigid frames and chevron bracings. The project is located in UBC Seismic Zone No. 3. The gravity and lateral loads were carried to the new foundation by adding columns to the existing concrete construction. New piles were driven under the new footings. The project architect was Zimmer, Gunsul, Frasca Partnership and the structural engineer of record was KPFF Engineers of Portland, Oregon.

The General Contractor was Baugh Construction Inc. of Seattle, Washington. The structural steel was fabricated by Fought and Company and the steel erection was contracted to Midwest Steel Erectors of Detroit. The steel erection started about a few months earlier and was progressing well without any significant incident, though not in compliance with the general recommendations of the structural consultant retained by the steel erector. The steel erector retained a structural consultant, NAVA Contracting & Engineering Inc., to recommend and advise the measures to be taken and procedures to be followed by the steel contractor to safeguard the existing parking garage in the event of high wind and an earthquake during the construction. This was done because the existing parking garage was kept open for use by the patrons during construction. The steel erector's consultant had essentially recommended that only two levels of parking be constructed at a time. Âfter all the steel columns, beams and girders are erected, proceeding from west to east, they be diagonally braced in each bay and all permanent connections are made. Floor metal decks be placed and concrete be cast. After completing the structural integrity of the two levels as designed for the permanent conditions, then the steel erector should proceed higher and begin the erection of the next two higher levels in the same manner (copy of the typical bracing detail prepared by the Consultant Engineer attached). For unexplained reasons, the steel contractor did not follow his consultant's recommendations. At the time of the incident, the north half of the parking garage structure and a few bays on the west side of the parking garage were completed up to the highest seventh level, although all beam to beam and beams to column connections were not permanently secured. In addition, one bay on the east side adjoining to the collapsed bay was also erected up to the seventh level. On the southern half, the steel erector started the erection on the east face.

On July 31, 1997, the erection of the steel in the east bay between column lines P3 and P0 and south of the column line PE was started. The structural steel north of line PE was already erected up to the seventh level, as mentioned earlier. The typical procedure of the steel erection was to erect the columns, then the girders running in the north-south direction, followed by the filler beams in the east-west direction and then chevron braces on the east face. The framing south of column line PE was already erected up to the third level earlier. See Figures 1 to 6 for the structural members erected above the third level on July 31, 1997 up to the time of the incident. All fabrication and identification marks are shown on the figures.

Two columns, P0-PC1 and P0-PD1, were erected over the bottom splice located a few feet above the third level. The third column P3-PD was placed over the third level with anchor bolts. The columns were also spliced at locations a few feet above the fifth level. The columns at P0-PC1 and P0-PD1 extended above the seventh level to frame the rooftop of the elevator shaft. Typically, at the column splices were two erection bolts connecting the splice plates which were to be later welded to the column web. The column flanges would later receive full penetration groove welds. However, at the time of the incident, there were only two erection bolts. Wedges were also placed, two at each flange. See Figure 7. The column at P0-PC1 consisted of two pieces, marked 2225A and 2225B, 23'-1" and 38'- ½" in height respectively. The column at P0-PD1 also consisted of two pieces marked 2226A and 2226B and were also of the same lengths as the previous columns mentioned. The column at P3-PD consisted of two pieces marked 2219A and 2219B, and were 19'-5" and 26'-7 5/8" long respectively. There were no beams framing in the east-west direction to the three columns. Figures 3 to 6 indicate the location of the beams framing at the time of the incident.

The above figures indicate that all three columns were unbraced in the east-west direction. For the entire length of 61'-2½" above the splice near the third level, the columns P0-PC1 and P0-PD1 were braced neither with any temporary erection bracings in the east-west direction nor with any beam framing directly to it at any level. The same was true for the column at P3-PD. The closest filler beam to column P0-PD1 and P3-PD was erected 10'-6" away at some levels.

All chevron bracings on the East face between P0-PC1 and P0-PE were erected up to the sixth level. The bracings between the sixth and seventh levels were to be installed when the incident occurred. As the crane hoisted one of the braces and was holding the load above the seventh floor a few feet from its final destination, the bay bounded by column lines P3 and P0 and column lines PC1 and PE collapsed in the easterly direction. All girders and filler beams above the third level fell. The columns located at P0-PC1 and P0-PD1 separated at the splice locations at the third level. The column located at P3-PD failed at its base where it was secured by anchor bolts. Three anchor bolts sheared and one was pulled out. See Figures 8 to 11.

At the time of the incident, there were four iron workers on or near the collapsed bay. Two were at the top girders at the seventh level and one at the top girders at the sixth level waiting for the brace to arrive. The fourth iron worker who escaped unhurt was reportedly on the fifth level beam on the east side. All were tied to lifelines placed on the girders. Reportedly, there were no activities going on minutes before the incident as they were awaiting the brace to arrive. Before the crane operator could maneuver the brace near its location for the three iron workers to position it at its location, the steel collapsed as discussed earlier.

Structural analysis and Discussion

Structural analyses of the steel framing bounded by columns lines PE, PC1, P0 and P3 were performed by using a finite element computer program, STAAD-III. The computer model consisted of 56 joints and 84 member elements, see Fig. 12. The bottom of the columns was assumed to be fixed at their bases at the third level. The east exterior columns were fabricated in two sections and spliced at locations of 3'-5 ½ " above the 3rd and 5th floor levels as discussed earlier, and the interior column at the column line PD also was spliced at the same elevation above the 5th floor level. Since the column splices were secured by temporary measures consisting of two erection bolts, a joint was created at the splice locations capable of resisting only approximately 10% of the column flexural capacity. All beams were modeled as hinged at each end. The north columns on line PE were modeled without any translational freedom in the north-south and east-west directions.

The physical dimensions of the structure and the member sizes were taken from the construction documents and shop drawings. No deviations were assumed from the contract drawings and shop drawings. Dead loads of the structure were considered by the computer program as uniformly applied loads for all members. Weights of workers who were standing at the 5th, 6th and 7th floor levels were applied as concentrated loads to each beam member. Loads from miscellaneous items such as tools carried by the workers were ignored. Wind data obtained from the National Weather Service (copy attached), indicated that wind speed at or about the time of the collapse was approximately 8 mph. The computations are, however, based on 10 mph wind speed.

The above analysis was performed based on the assumption that the structural framing was in fact plumb and square. This assumption will be discussed later in detail. The field observation made by the OSHA's investigating team following the incident revealed that the connections of the girders to the columns were made by two bolts with the exception of the north end of the girder marked 2890A which had three bolts. The connections of the east-west filler beams to the girder were all made by one bolt only at each end and the connections of the chevron bracings to the exterior girders were made by one bolt at the upper ends only. These connections were assumed to be pinned in the computer model and the members were also assumed to be leveled. See Fig. 13 to 18 for number of bolts at each connection.

Of significance to this report was the computation of the critical buckling loads of the columns and the maximum allowable flexural capacity of the column due to lateral torsional buckling considerations. Difficulties were encountered to determine the unbraced length and critical axial and flexural moment capacities of the columns under consideration due to the presence of two splices in the east columns and one splice in the west column. Theoretical solutions are rare and actual test data are lacking. Manual computations were, therefore, performed to compute the critical buckling load of the columns in accordance with the Load Resistance Factor Design (LRFD) of the American Institute of steel construction (AISC), assuming that there were no splices. The actual critical loads of the columns would be lower than the computed values due to presence of the splices. In computations, the load factor and resistance factors were considered as 1.0.

At the application of the wind load of 10 mph and considering that the splice is only capable of resisting 10% of the column flexural capacity, the axial load and the bending moment at the base of the column PD-P3 about its major axis were computed to be 24 kips and 164 k" respectively resulting in a combined stress of 2.0 ksi. Under similar assumptions, the column P0-PC1 was subjected to an axial load of 30 kips and a bending moment of 110 k" about its minor axis. The column P0-PD1 was subjected to an axial load of 60 kips and a bending moment of 100 k" about its minor axis. If the effects of the column splices are ignored, the critical buckling load and the nominal flexural strength of the column PD-P3 were estimated to be higher than the imposed load. The columns at P0-PC1 and P0-PD1 were W14x90 (non-compact due to the flange criteria). The nominal flexural capacity of the columns was computed to be 3,700 k" about the minor axis against the actual imposed moment of 100 to 110 k". If the splices are ignored, the critical buckling load would be higher than the load imposed upon them. With the splices, however, the critical buckling load and the nominal flexural capacity of the columns could not be accurately determined due to lack of test data.

The foregoing calculations were performed on the premise that the steel columns were truly vertical. This assumption facilitated the computations but site conditions do not support this premise. The steel columns and the beams were mostly erected on the morning of the day of the incident and the bay which collapsed was yet to be squared and plumbed. It cannot therefore be expected that the columns would truly be vertical. The expected out of plumbness of the columns would be based on the fact that the columns P0-PC1 and P0-PD1 were spliced at two locations. At each location, the splices were secured by two erection bolts. Four wedges were placed between the upper and lower column flanges. The erection bolts and the wedges were temporary measures until the bay could be squared and plumbed. The column flanges at the splice location would then be welded by full penetration groove welds. The third column P3-PD was supported on the third level by four anchor bolts and shim plates. There was no grout under the base plate. The column P3-PD was also spliced at a location above the fifth level and secured in a similar manner. Most important, there were no beams framing in the east-west direction directly to any of the three columns at any level. There were few filler beams at certain levels running in the east-west direction, as noted earlier. The closest filler beam was at least 10'-6" away which could not be expected to brace the column in the east-west direction. Also, the filler beams were connected with only one bolt at each end. The steel erector did not provide any temporary bracings in the east-west direction to any of the columns at any level.

All the foregoing leads to the fact that the columns P0-PC1, P0-PD1 and P3-PD were neither plumbed nor secured nor braced. They were essentially in a state of instability, ready to collapse at the application of a gravity and/or lateral load above the threshold capacity of the columns. It is difficult to ascertain with a high degree of accuracy the failure load of the columns because of a number of variables and unknown factors. The column P0-PC1 and P0-PD1, each 61'-2 1/2" high, were unbraced about their minor axis supported on a splice with yet another splice at their mid height. The splice, though acting as hinges, had some minimal moment capacity which the steel erector could not depend upon. Such minimal capacity is always disregarded and never counted upon to provide stability to the erected structure. ANSI Standard A 10.13-1989 states in section 11.1.9 that "all structural steel framing shall have its structural integrity protected promptly upon erection by bracing adequate to resist horizontal forces such as wind and reactions by erection equipment." (Emphasis ours). Copy of the ANSI Standard attached.

CONCLUSIONS:

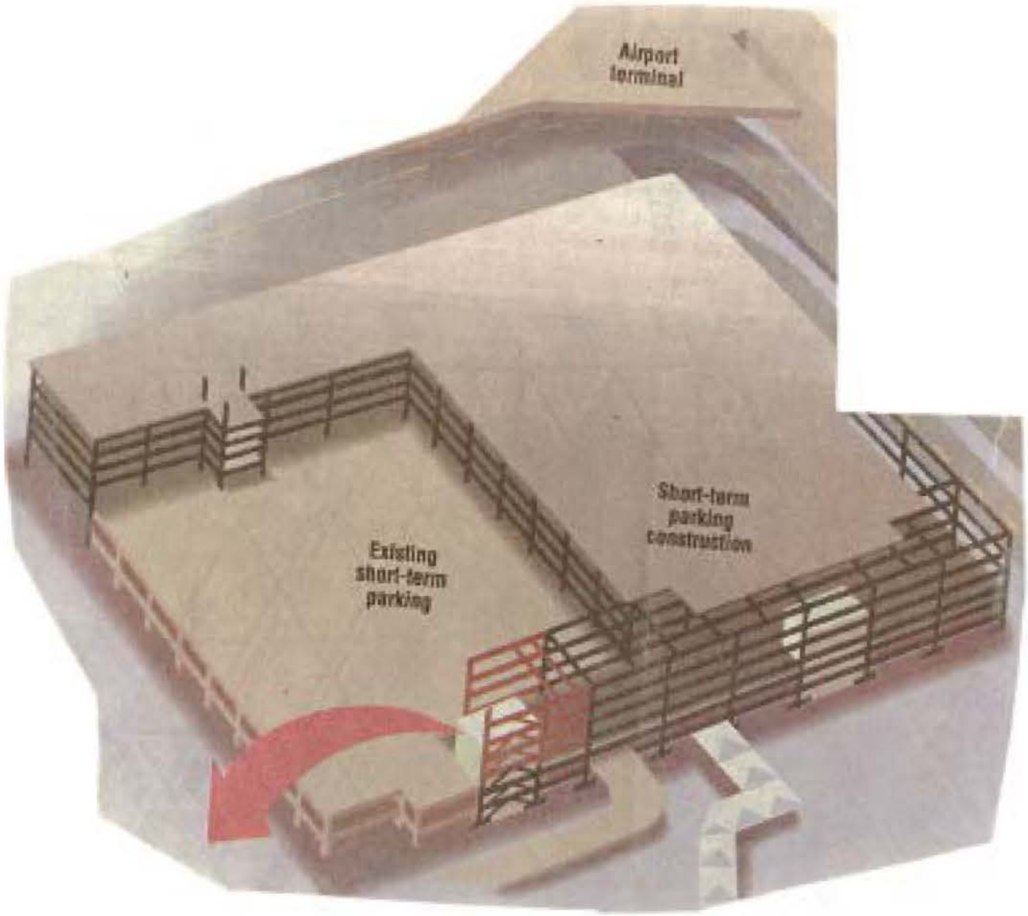
Based upon the above, we conclude that:

1. The steel erector did not comply with the sequence of erection and procedures recommended by its consultant.

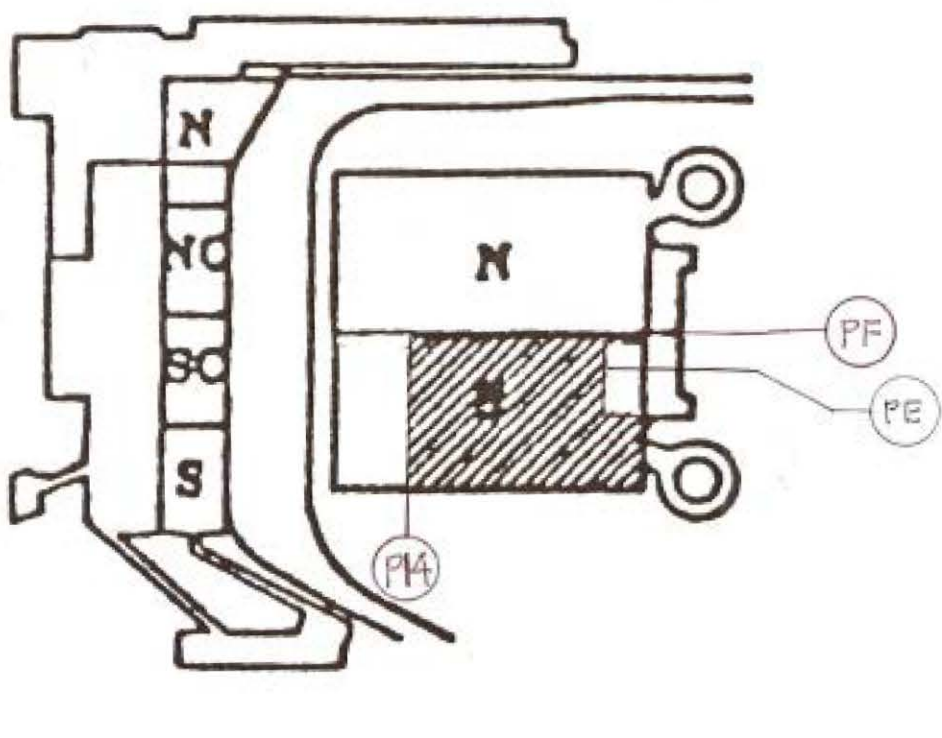
2. The steel erector failed to provide temporary bracing and guys to the erected steel to secure them against any lateral load due to wind and/or any incidental construction load as per the standard steel industry practice and as required by the American Institute of Steel Construction and American National Standards Institute.

3. As the collapsed unplumbed steel frame was not provided with any temporary bracing, it was rendered unstable and was in an imminent danger of collapse.

4. Filler beams were provided with only one erection bolt at each end in violation of the OSHA's minimum requirement of two bolts at each end.

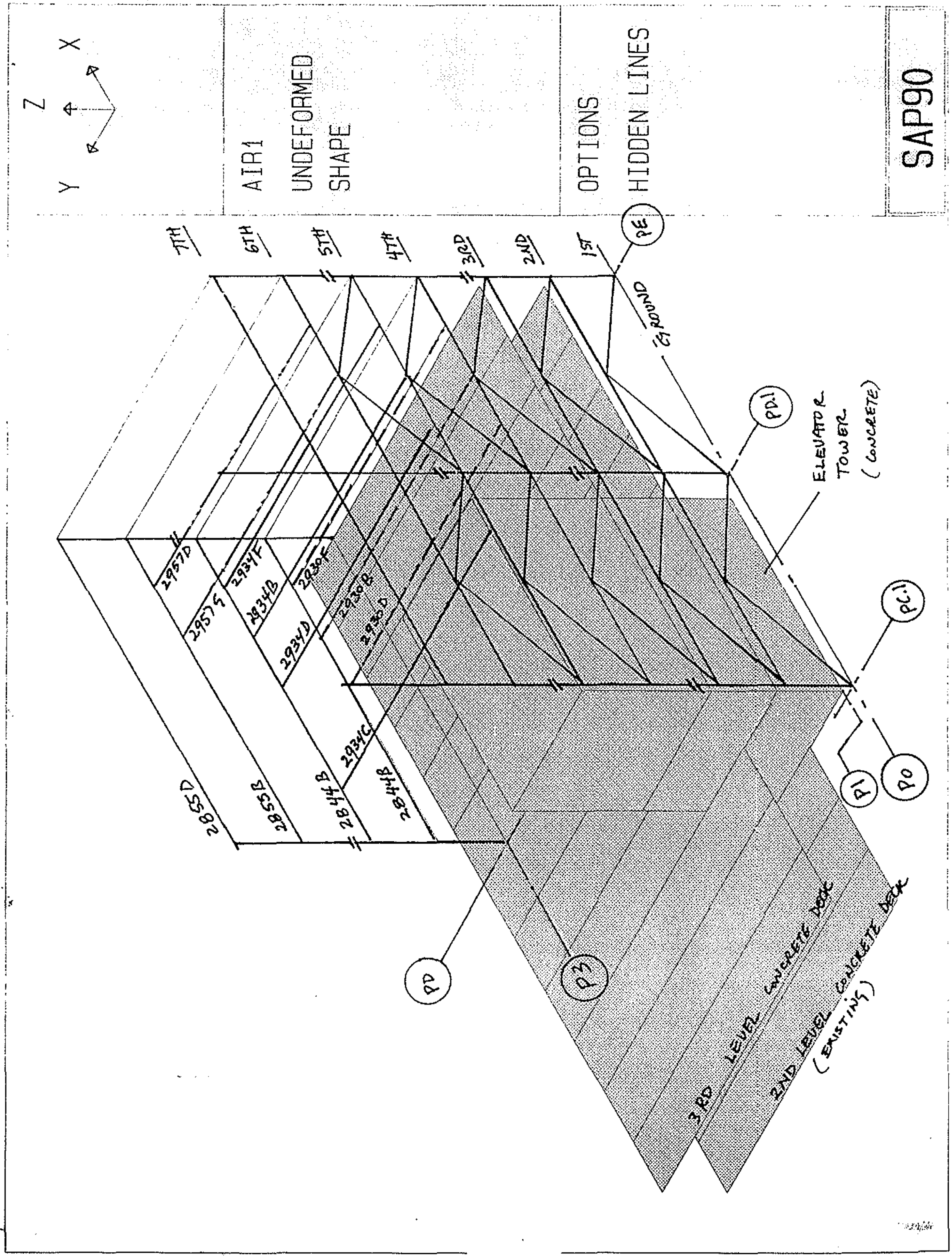


Airport terminal
Short-term parking construction
Existing short-term parking



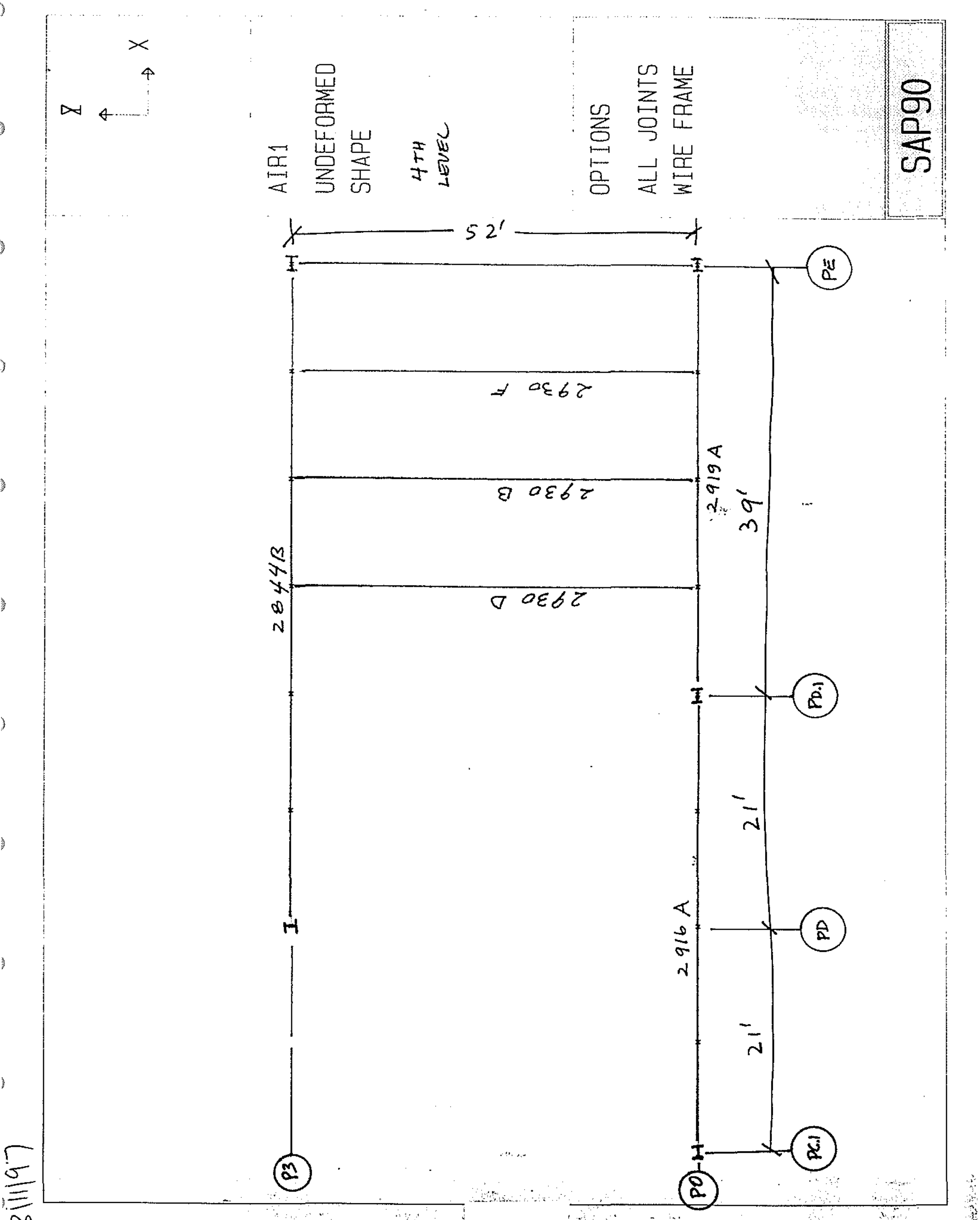
N
N
S
S
N
PF
PE
P14

Fig. 1


SAP90
AIR1
UNDEFORMED
SHAPE
OPTIONS
HIDDEN LINES
Z
Y
X
7TH
6TH
5TH
4TH
3RD
2ND
1ST
GROUND
PE
PD.1
PC.1
P1
P0
PD
P3
ELEVATOR
TOWER
(CONCRETE)
2957D
2957S
2934F
2934B
2934D
2930F
2930B
2930D
2934C
2855D
2855B
2844B
2844B
3RD LEVEL CONCRETE DECK
2ND LEVEL CONCRETE DECK
(EXISTING)

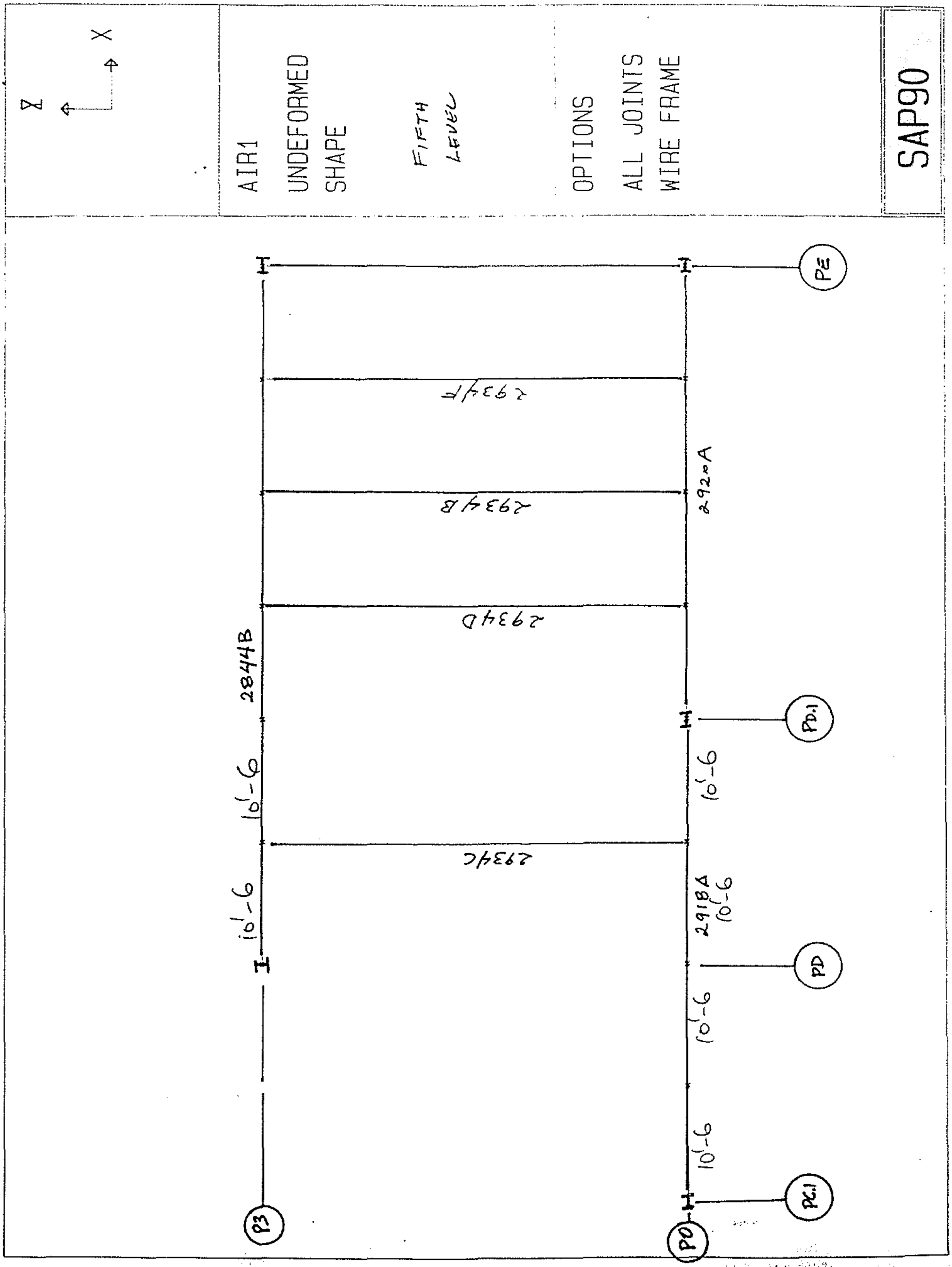
8/11/97

Fig. 2



Y
X
AIR1
UNDEFORMED
SHAPE
4TH
LEVEL
OPTIONS
ALL JOINTS
WIRE FRAME
SAP90
52'
PE
2930 F
2930 B
2919 A
39'
2844B
2930 D
PD.1
21'
2916 A
PD
21'
PC.1
P3
PO
8/11/97

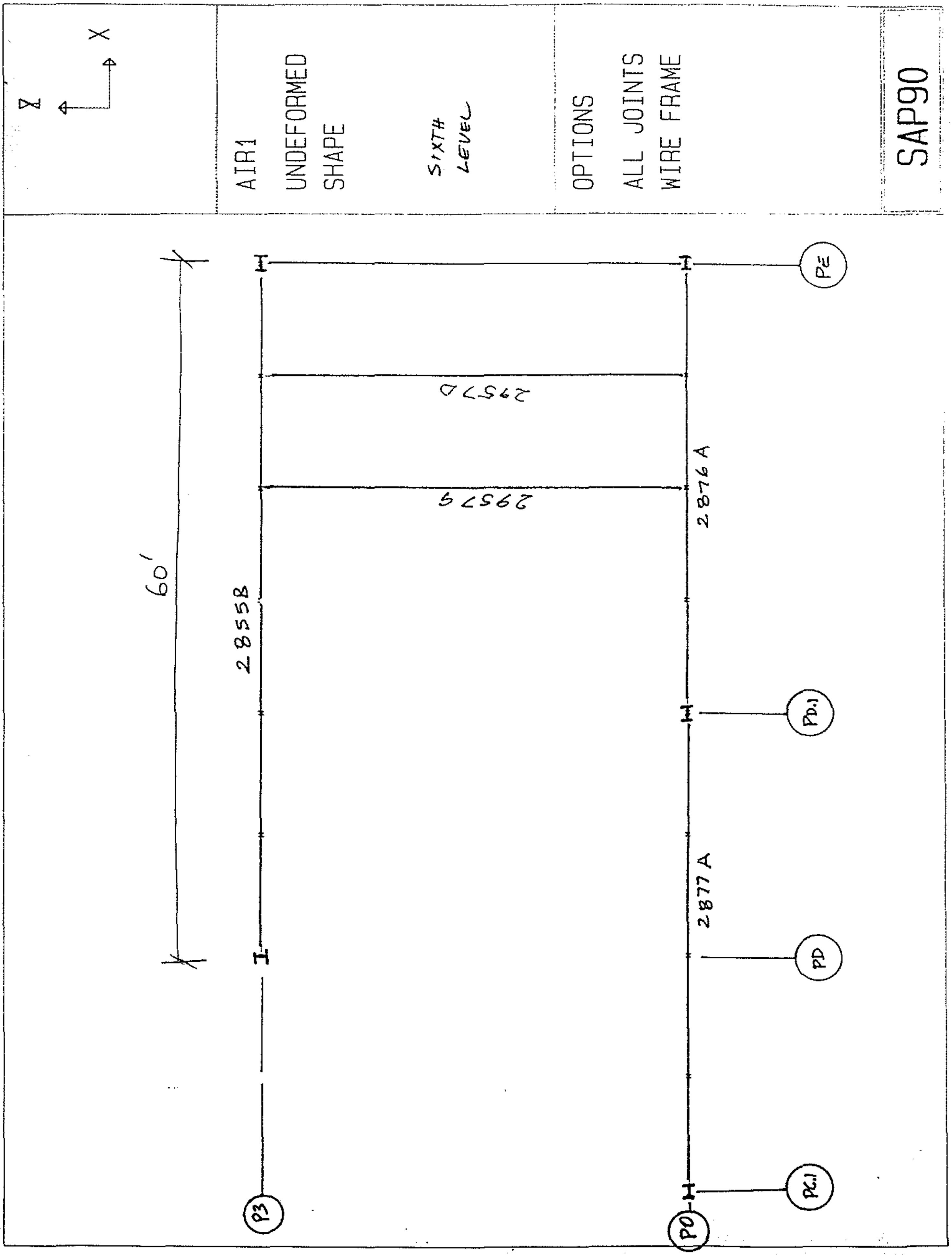
Fig. 3



Y
X
AIR1
UNDEFORMED
SHAPE
FIFTH
LEVEL
OPTIONS
ALL JOINTS
WIRE FRAME
SAP90
PE
2934F
2920A
2934B
2934D
2844B
10'-6
10'-6
PD.1
10'-6
2934C
10'-6
2918A
10'-6
PD
10'-6
10'-6
PC.1
P3
PO

Fig. 4

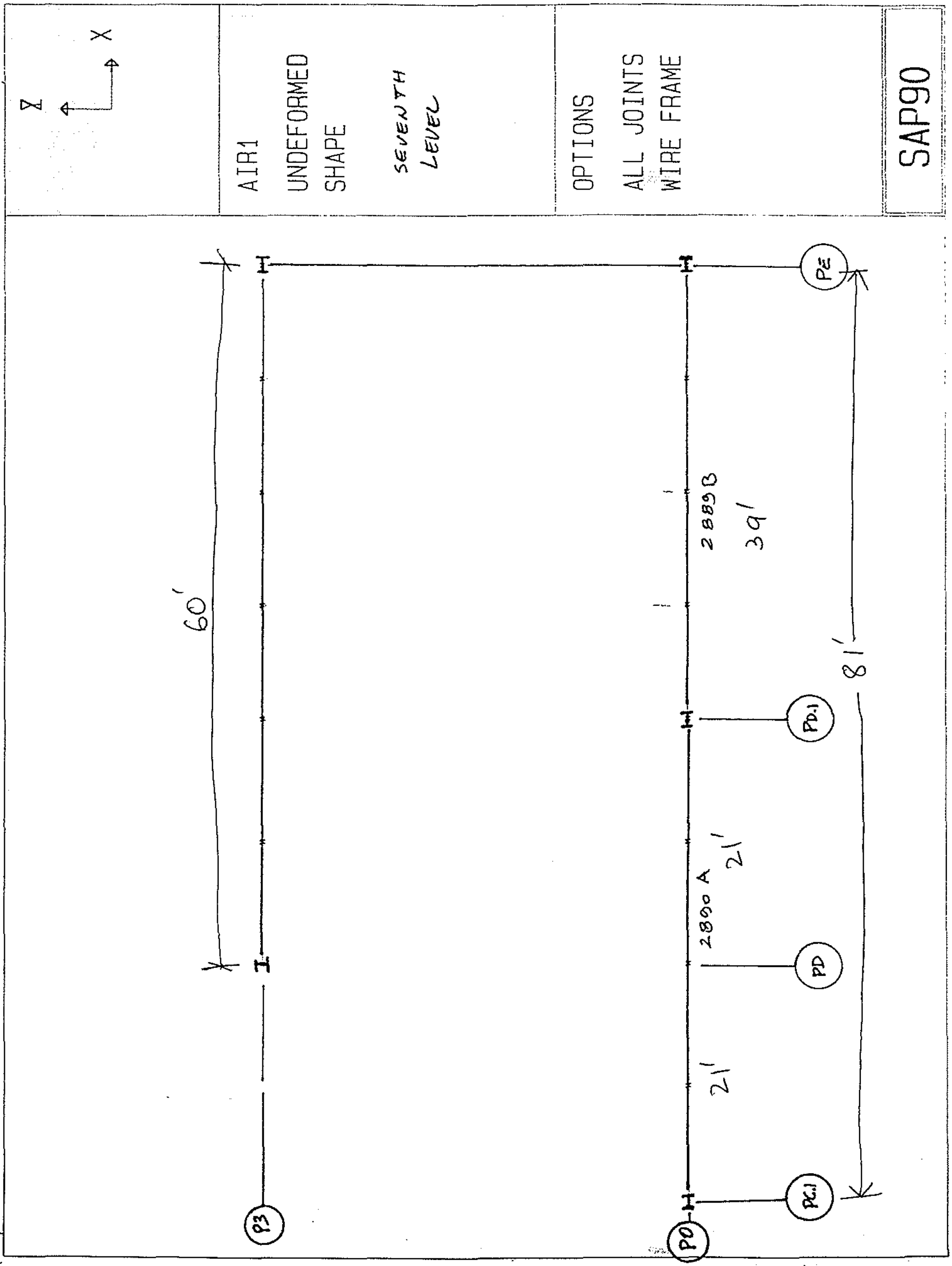
8/12/97



X
Y
AIR1
UNDEFORMED
SHAPE
SIXTH
LEVEL
OPTIONS
ALL JOINTS
WIRE FRAME
SAP90
60'
2855B
2957D
2957S
2876A
2877A
PE
PD.1
PD
PC.1
P3
P0

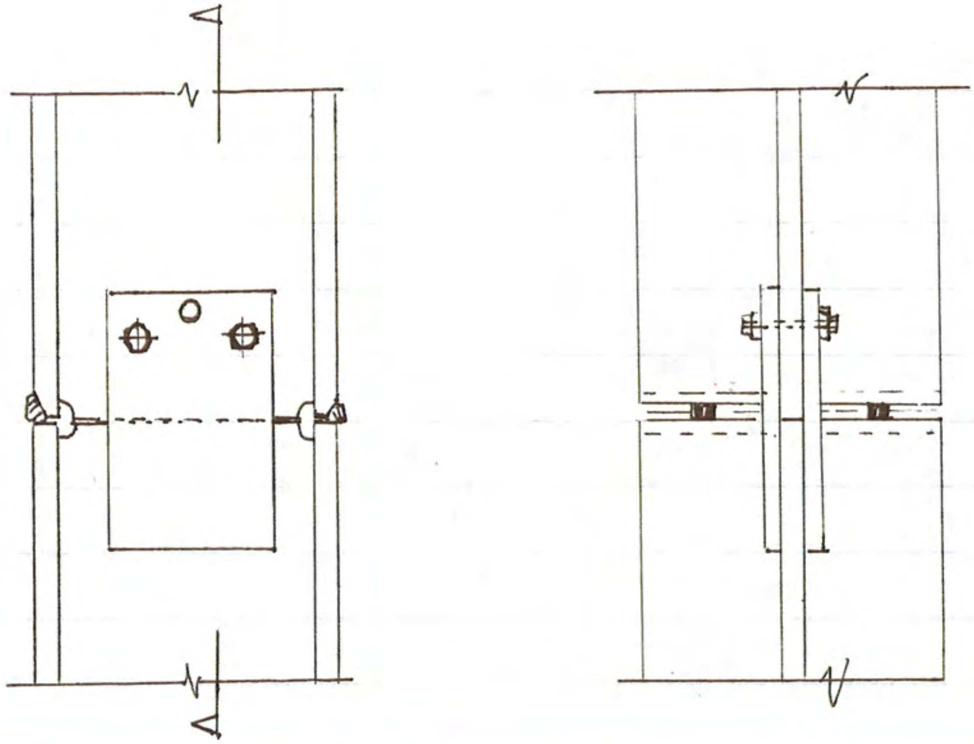
8/11/97

Fig. 5



Y
X
AIR1
UNDEFORMED
SHAPE
SEVENTH
LEVEL
OPTIONS
ALL JOINTS
WIRE FRAME
SAP90
60'
2889 B
39'
81'
2890 A
21'
21'
P3
PO
PC.1
PD
PD.1
PE

Fig. 6



Column Splice Detail

Fig. 7

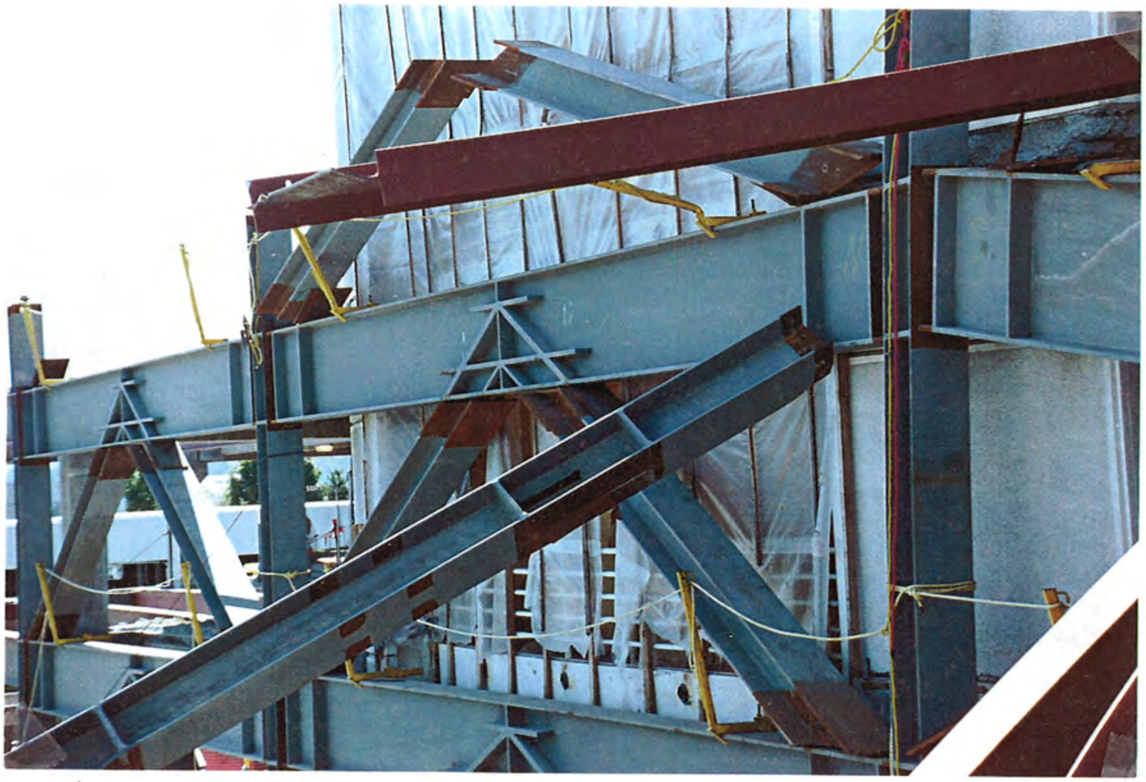


Fig. 8

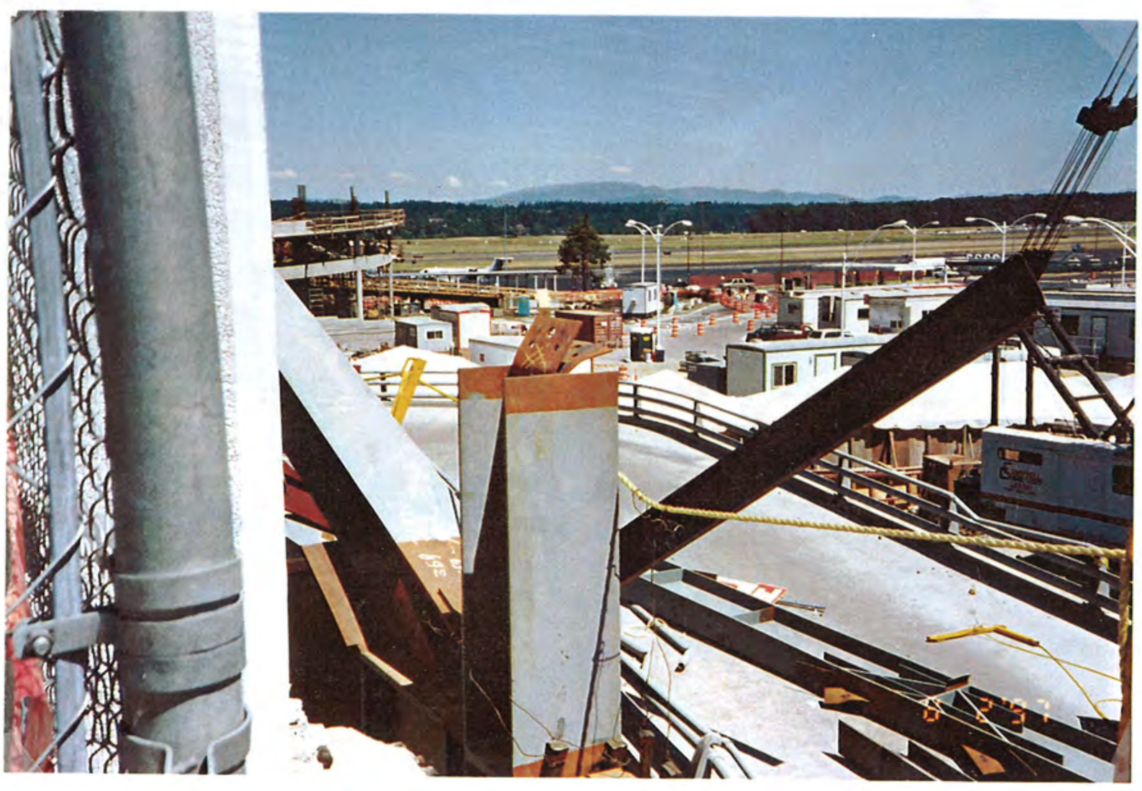


Fig. 9

←2219 A&B

Fig. 10

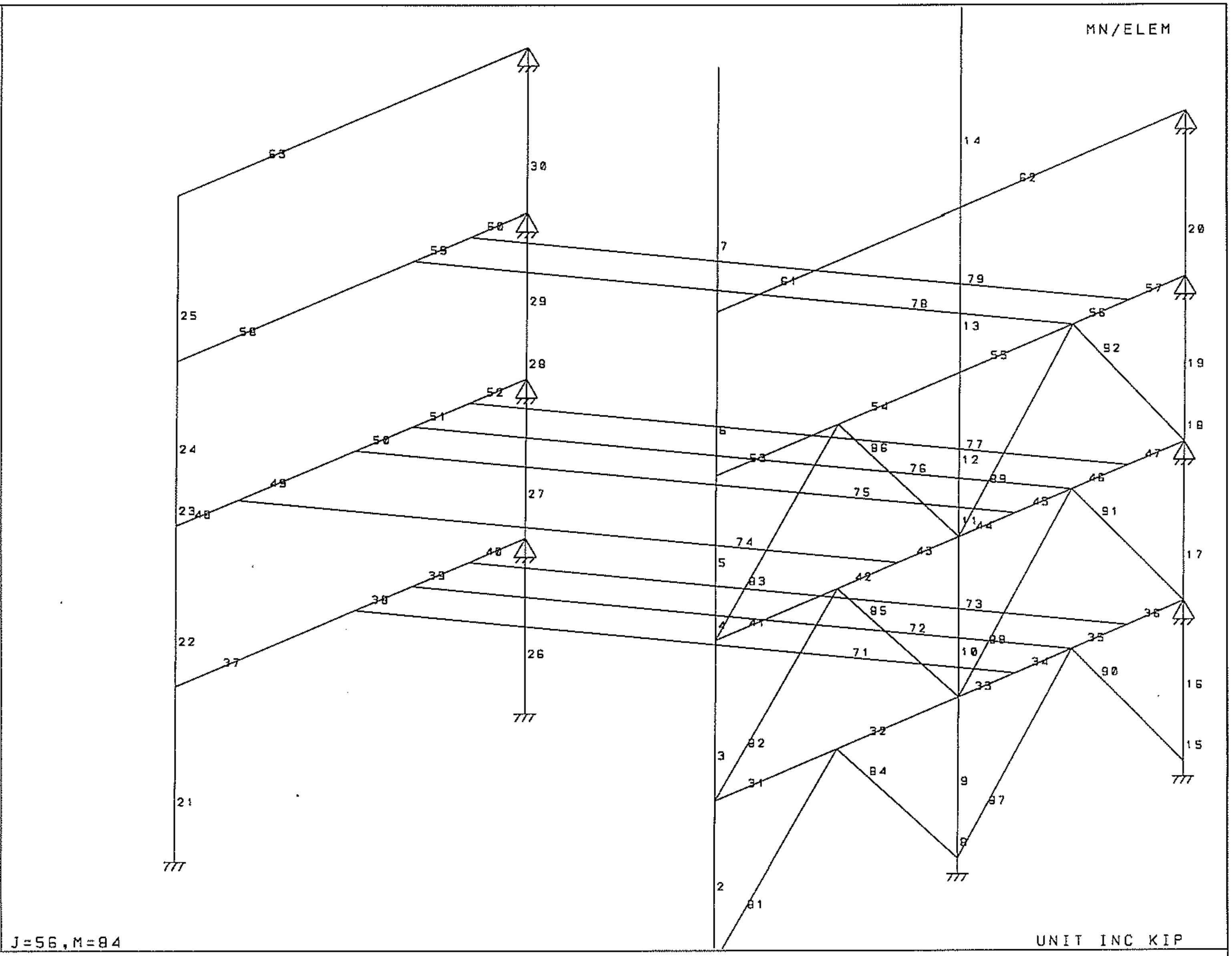
Fig. 11

User ID: US dept. of Labor/OSHA

Fig. 12

STRUCTURE DATA

TYPE = SPACE
NJ = 56
NM = 84
NE = 0
NS = 0
NRJ= 13
NL = 4
XMAX= 972.0
YMAX= 770.4
ZMAX= 624.0



S T A A D   P O S T - P L O T (REV: 22.3)   DATE: DEC 30, 1997

TITLE: - PORTLAND AIRPORT (AIRPORT11)

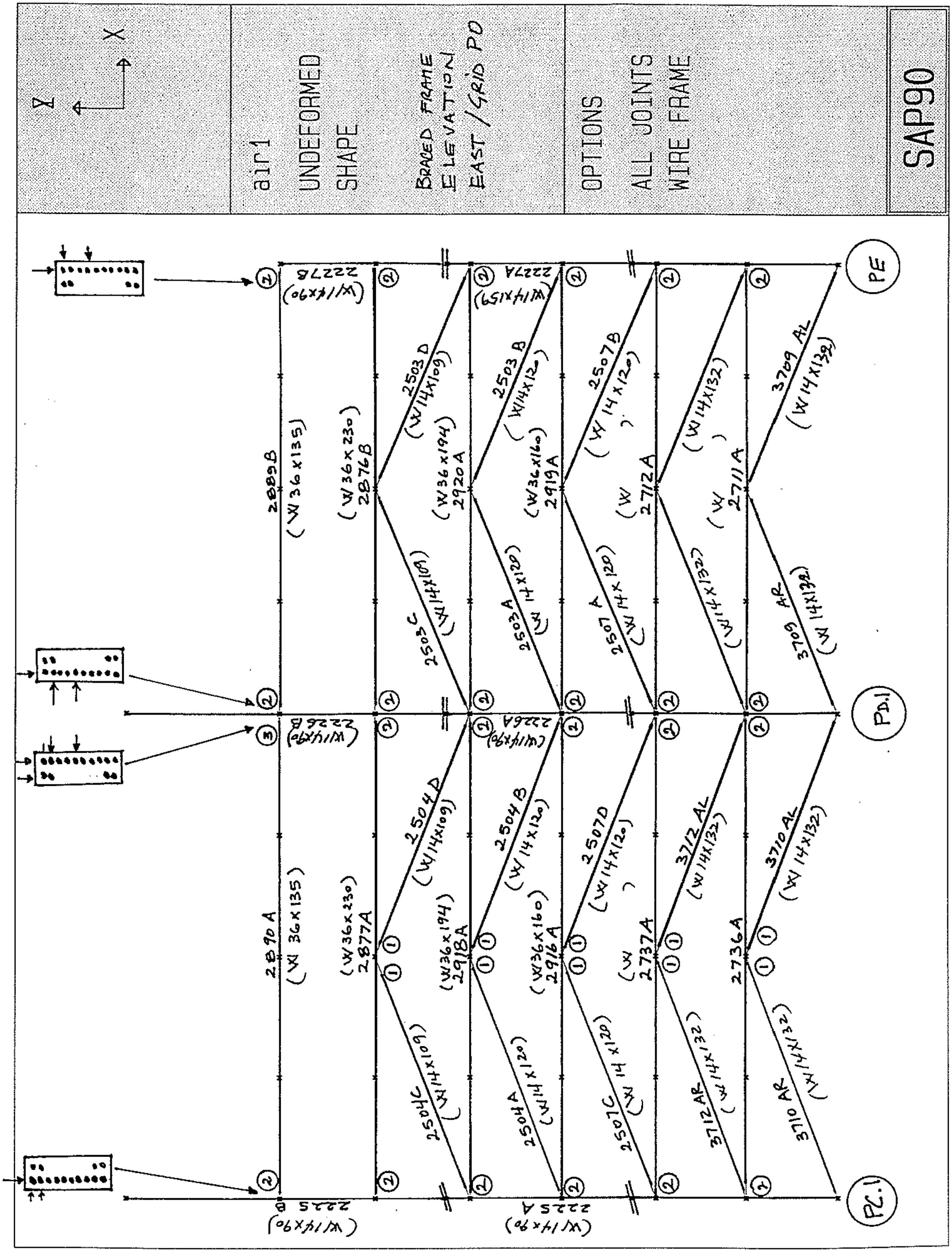

air1
UNDEFORMED
SHAPE
BRACED FRAME
ELEVATION
EAST / GRID PD
OPTIONS
ALL JOINTS
WIRE FRAME
SAP90
PE
PD.1
PC.1

Fig. 13

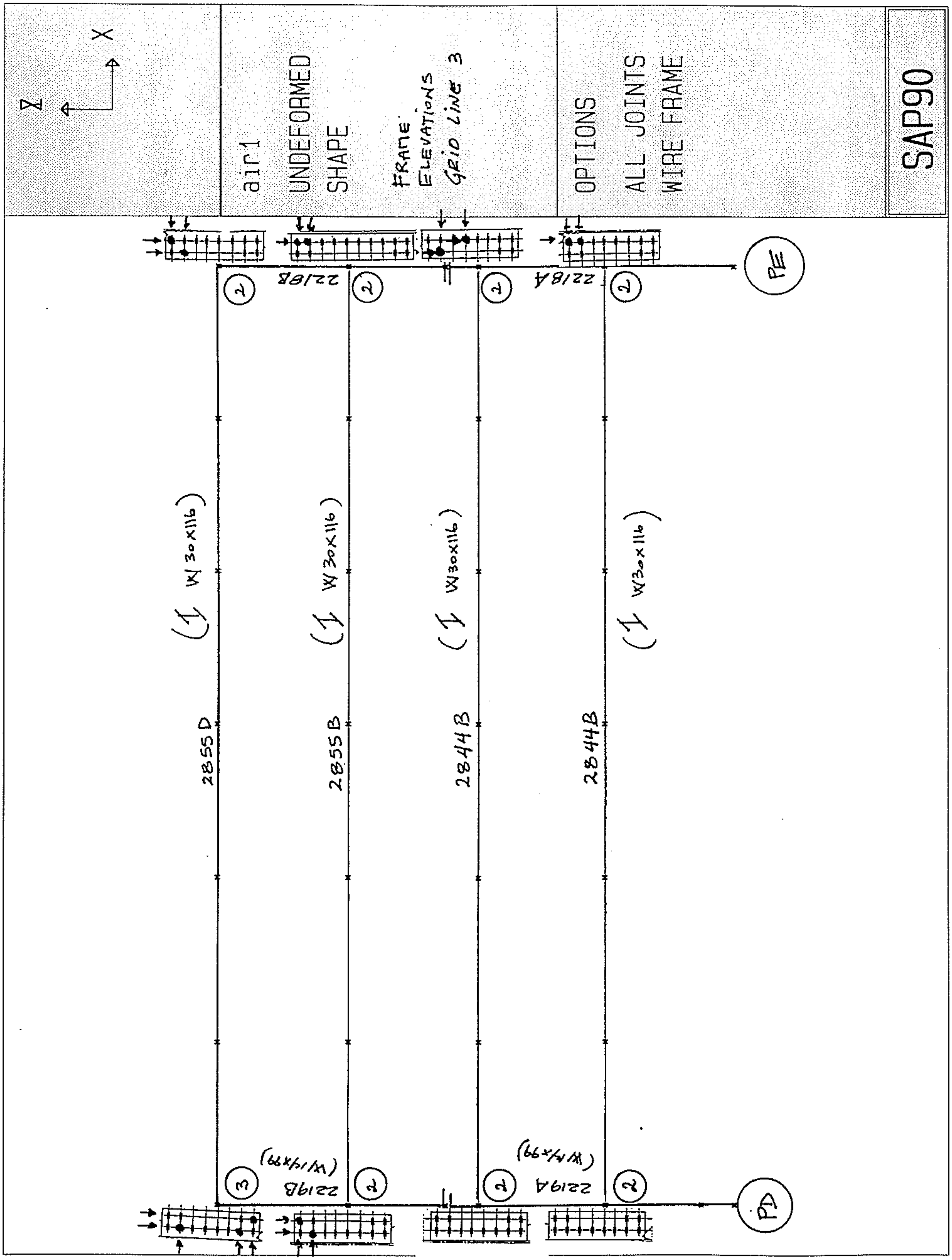
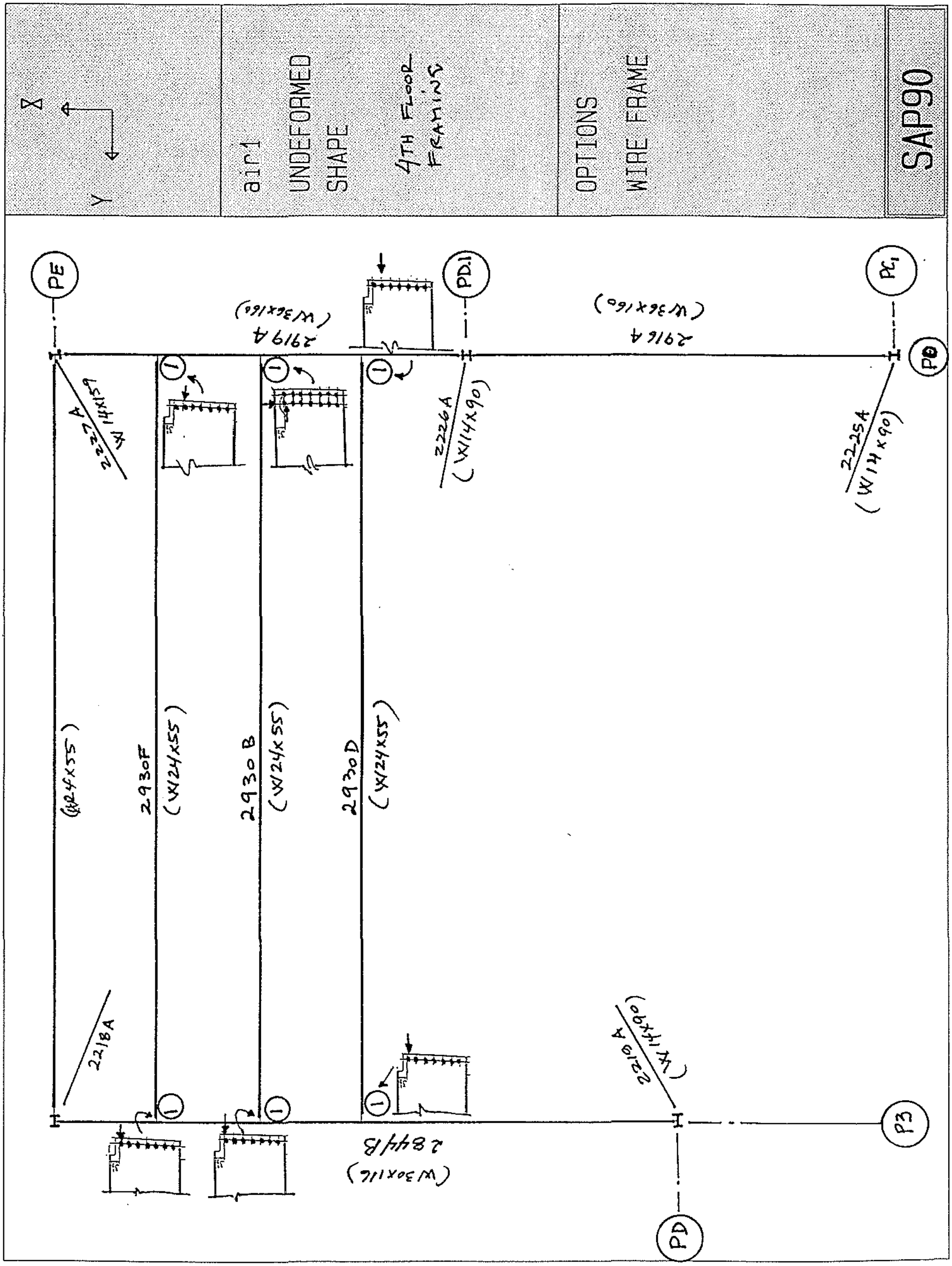

Y
X
air1
UNDEFORMED
SHAPE
FRAME
ELEVATIONS
GRID LINE 3
OPTIONS
ALL JOINTS
WIRE FRAME
SAP90
2218B
2218A
2219B
2219A
(W14x99)
(W14x99)
PE
PD
W30x116
W30x116
W30x116
W30x116
2855D
2855B
2844B
2844B

Fig. 14



X
Y
air1
UNDEFORMED
SHAPE
4TH FLOOR
FRAMING
OPTIONS
WIRE FRAME
SAP90
PE
PD.1
PC1
2919A
(W36x166)
2916A
(W36x160)
2227A
W14x159
2226A
(W14x90)
2225A
(W14x90)
(W24x55)
2930F
(W24x55)
2930B
(W24x55)
2930D
(W24x55)
2218A
2210A
(W14x90)
2844B
(W30x116)
PD
P3

Fig. 15

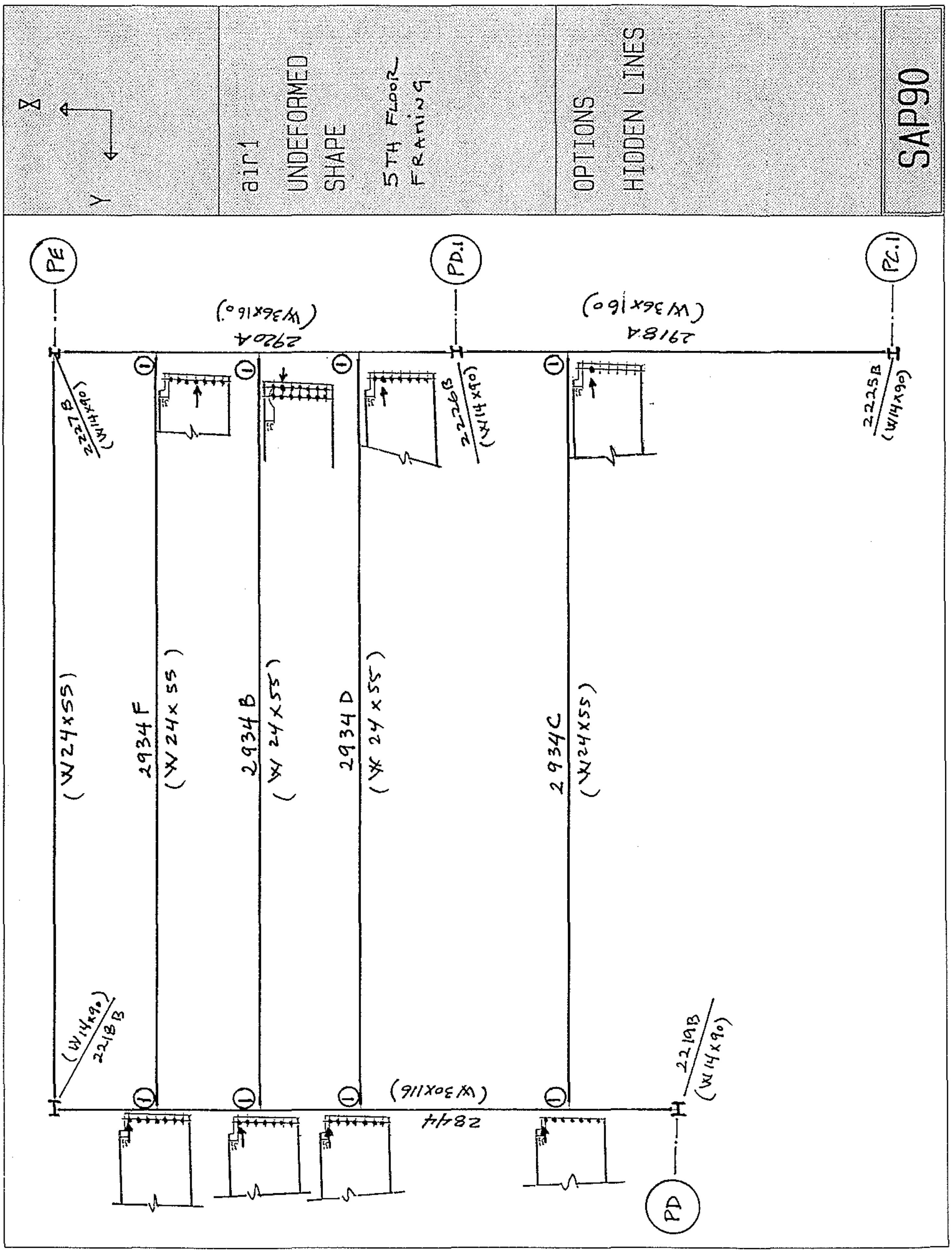

X
Y
air1
UNDEFORMED
SHAPE
5TH FLOOR
FRAMING
OPTIONS
HIDDEN LINES
SAP90
PE
PD.1
PC.1
PD
2920A
(W36x160)
2918A
(W36x160)
2227B
(W14x90)
2226B
(W14x90)
2225B
(W14x90)
(W24x55)
2934F
(W24x55)
2934B
(W24x55)
2934D
(W24x55)
2934C
(W24x55)
(W14x90)
2218B
2219B
(W14x90)
2844
(W30x116)

Fig. 16

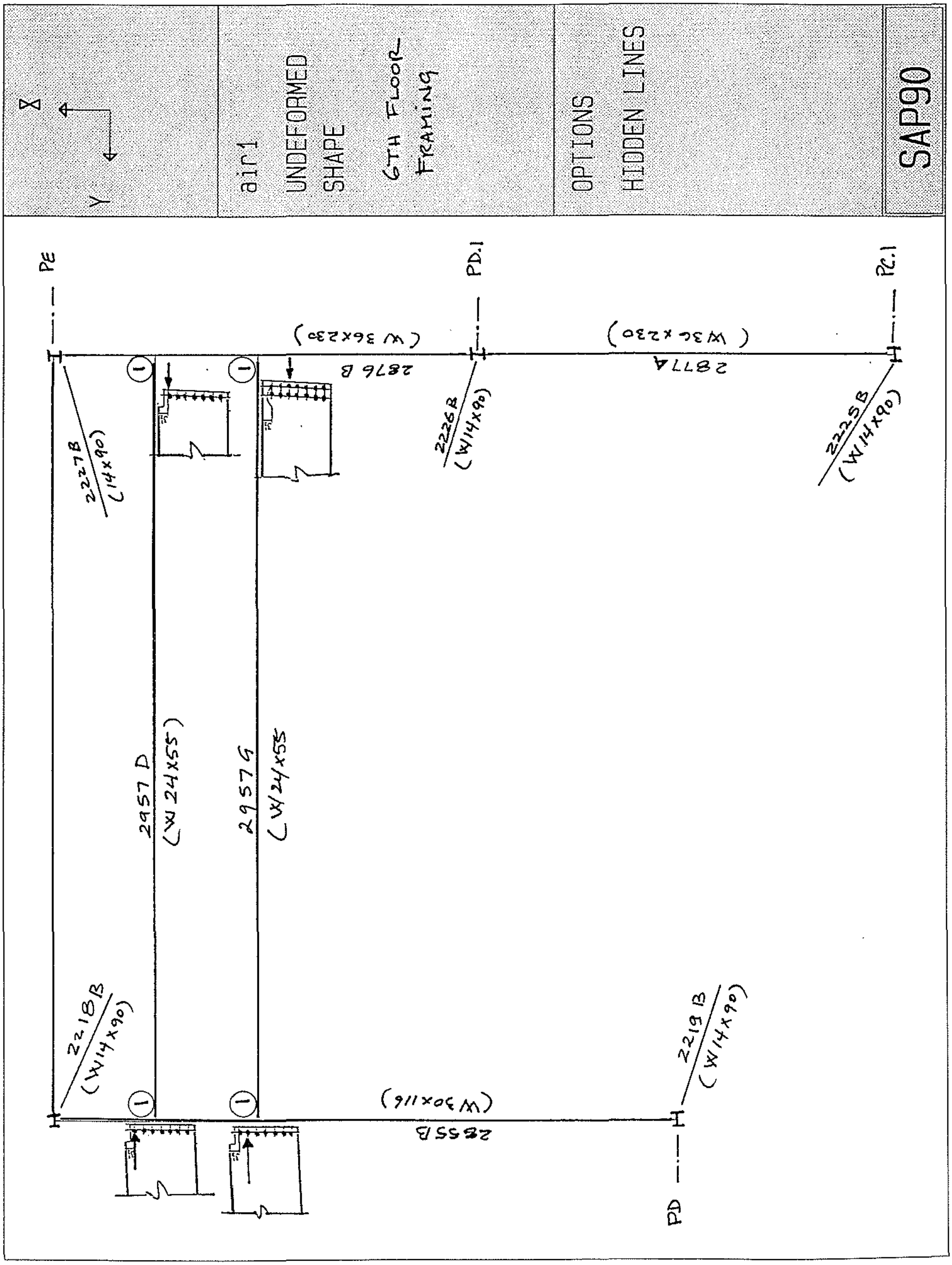
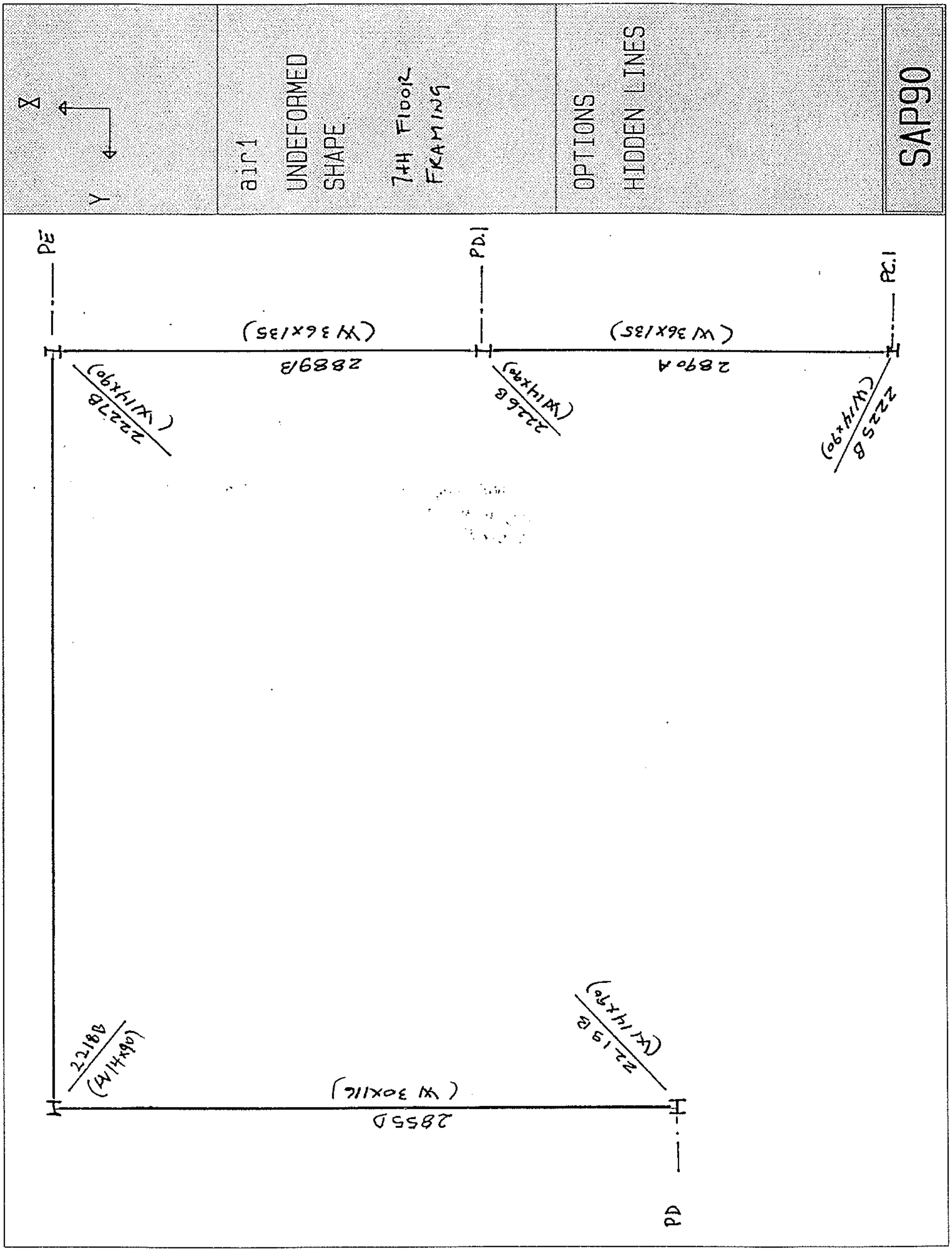

X
Y
air1
UNDEFORMED
SHAPE
6TH FLOOR
FRAMING
OPTIONS
HIDDEN LINES
SAP90
PE
PD.1
PC.1
PD
(W 36x230)
2876 B
(W36x230)
2877A
2227B
(14x90)
2226B
(W14x90)
2225B
(W14x90)
2957 D
(W 24x55)
2957 G
(W24x55
2218B
(W14x90)
2219 B
(W14x90)
(W30x116)
2855B

Fig. 17



X
Y
air1
UNDEFORMED
SHAPE
7TH FLOOR
FRAMING
OPTIONS
HIDDEN LINES
SAP90
PE
PD.1
PC.1
PD
2889B
(W36x135)
2890A
(W36x135)
2227B
(W14x90)
2226B
(W14x90)
2225B
(W14x90)
2218B
(W14x90)
2219B
(W14x90)
2855D
(W30x116)

Fig. 18

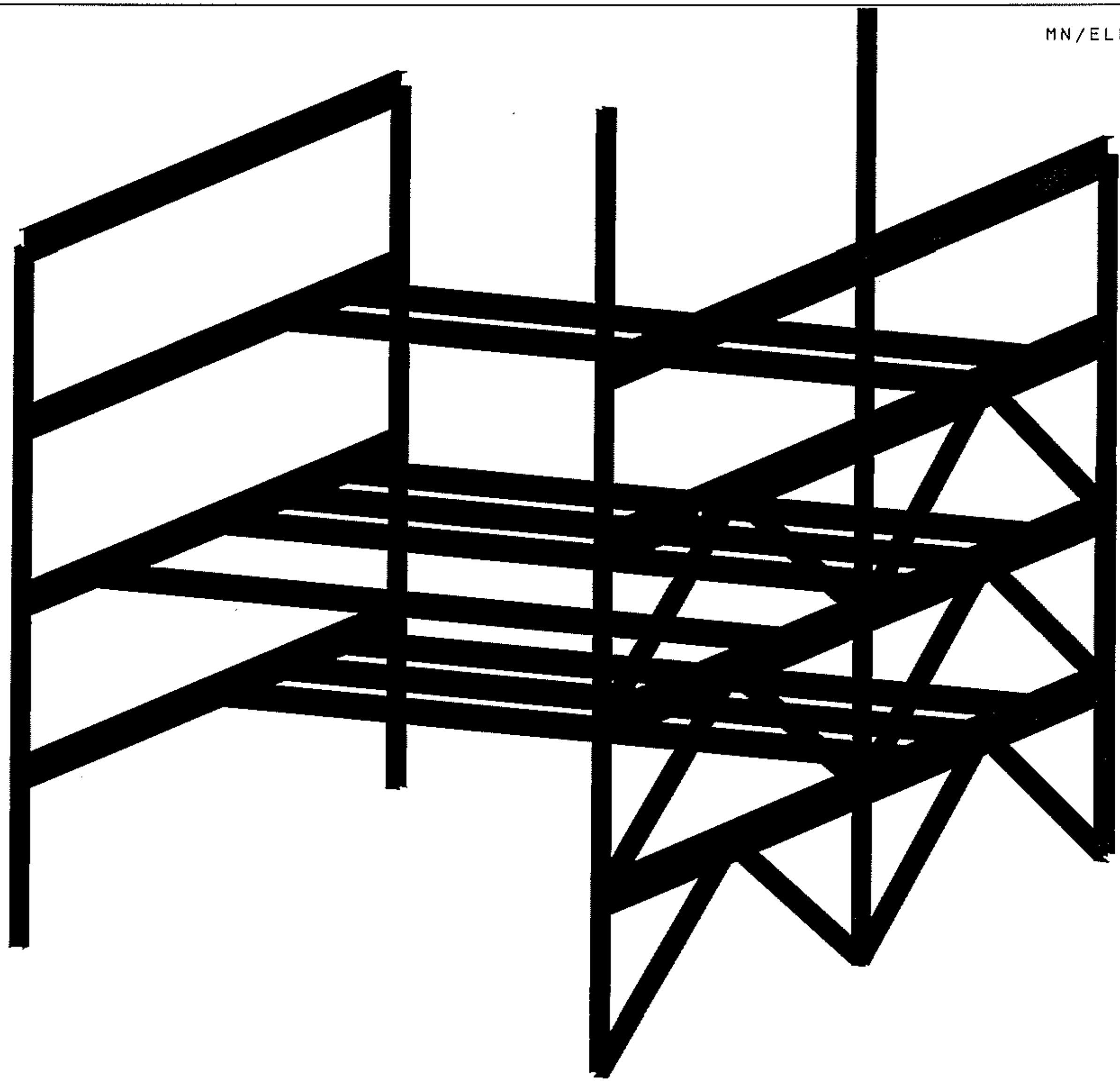
Computations

STRUCTURE DATA

TYPE = SPACE

NJ =	56
NM =	84
NE =	0
NS =	0
NRJ=	13
NL =	4
XMAX=	972.0
YMAX=	770.4
ZMAX=	624.0

MN/ELEM



J=56,M=84

UNIT INC KIP

S T A A D P O S T - P L O T (REV: 22.3)

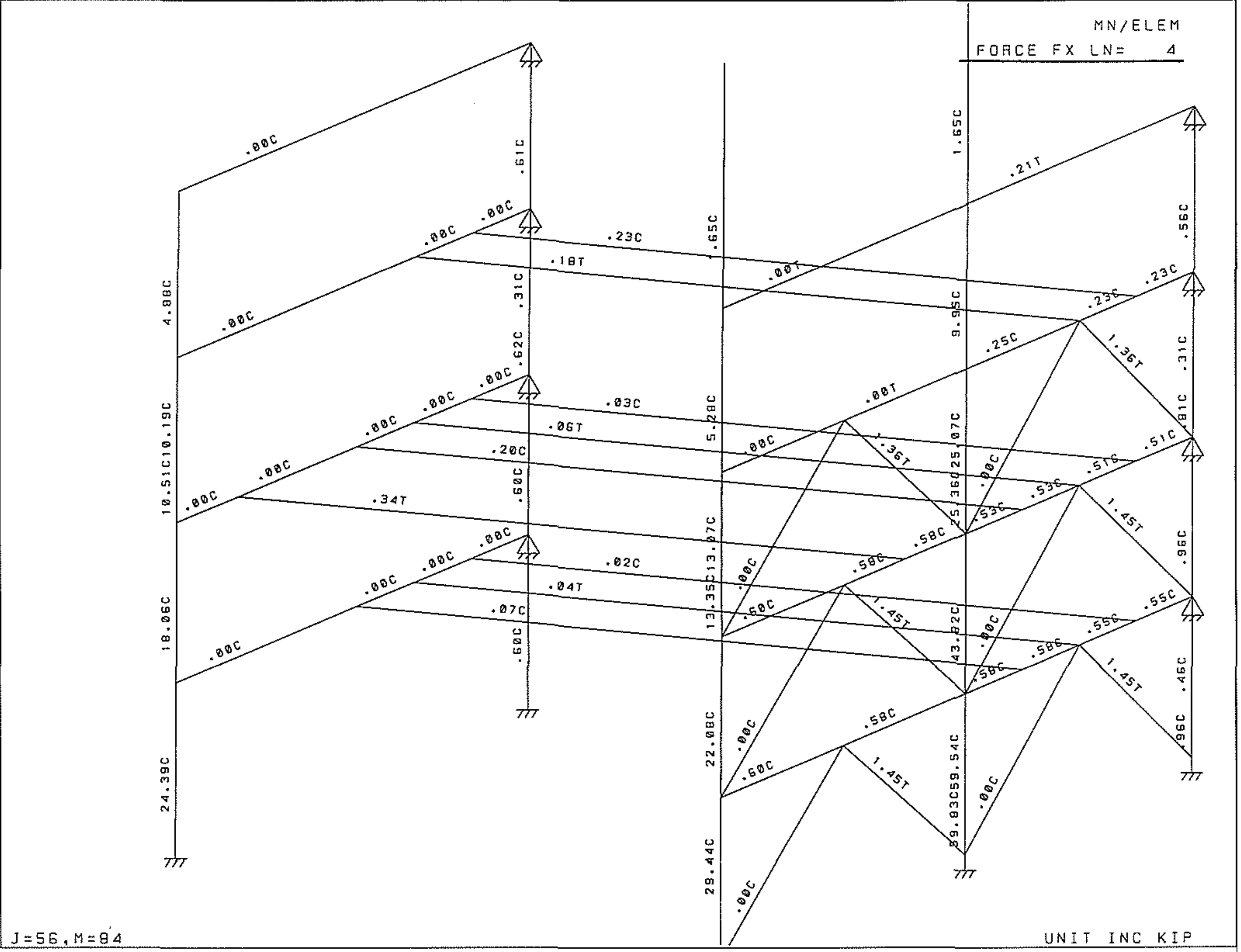
DATE: DEC 30, 1997

TITLE: - PORTLAND AIRPORT (AIRPORT11)

User ID: US dept. of Labor/OSHA

STRUCTURE DATA

TYPE = SPACE
NJ = 56
NM = 84
NE = 0
NS = 0
NRJ= 13
NL = 4
XMAX= 972.0
YMAX= 770.4
ZMAX= 624.0



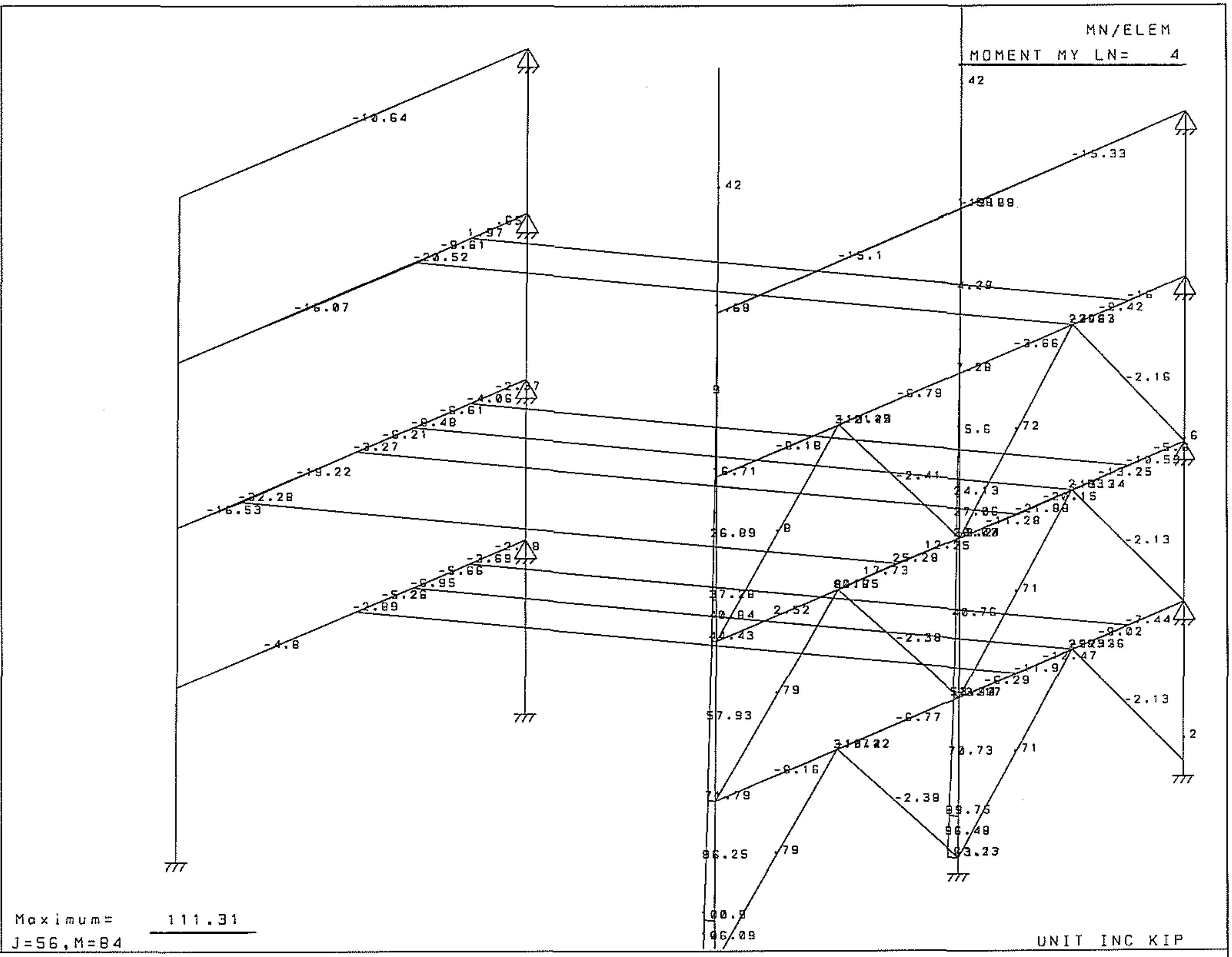
S T A A D P O S T - P L O T (REV: 22.3) DATE: DEC 30, 1997

TITLE: - PORTLAND AIRPORT (AIRPORT11)

User ID: US dept. of Labor/OSHA

STRUCTURE DATA

TYPE = SPACE
NJ = 56
NM = 84
NE = 0
NS = 0
NRJ= 13
NL = 4
XMAX= 972.0
YMAX= 770.4
ZMAX= 624.0



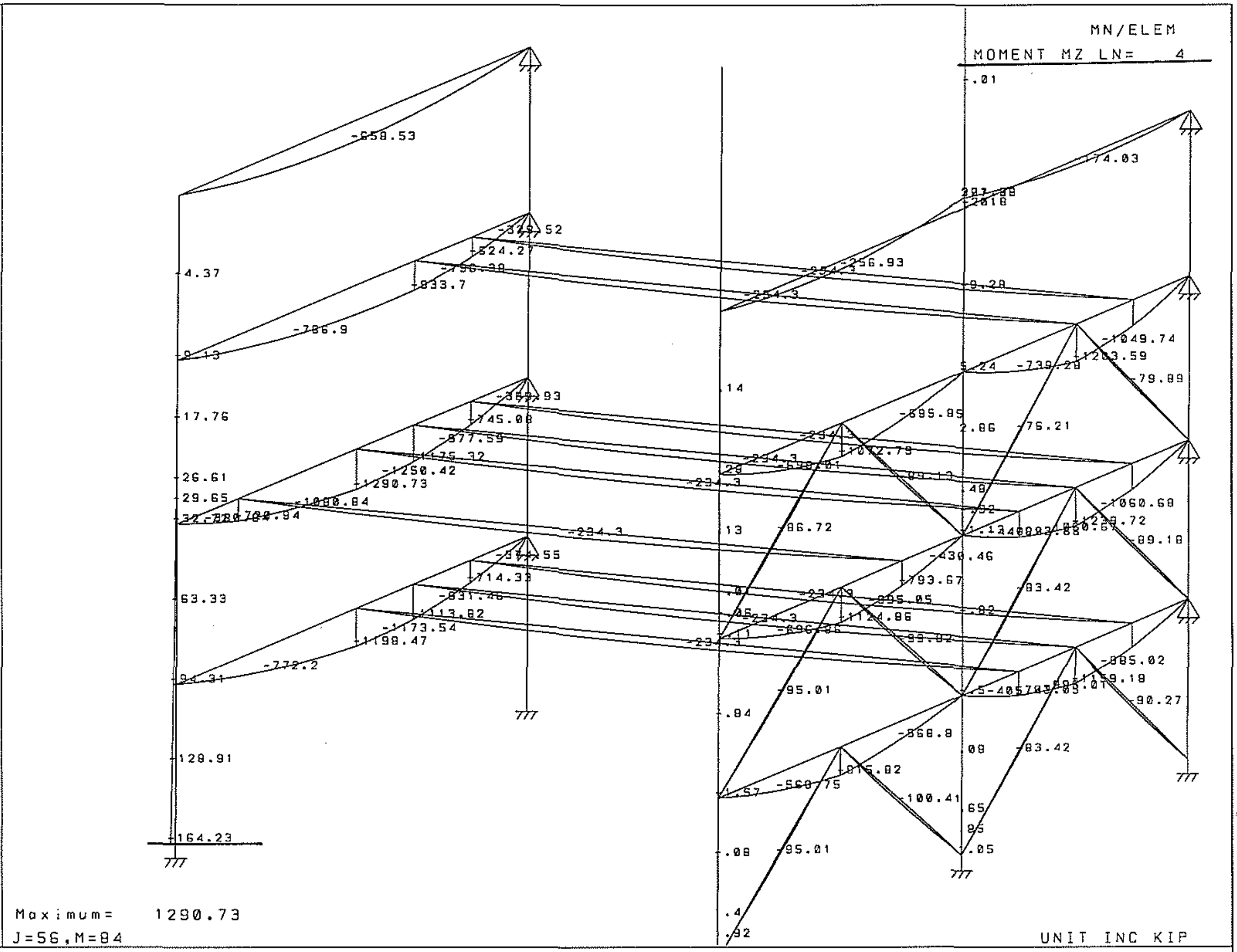
S T A A D P O S T - P L O T (REV: 22.3)

DATE: DEC 30, 1997

TITLE: - PORTLAND AIRPORT (AIRPORT11)

STRUCTURE DATA

TYPE = SPACE
NJ = 56
NM = 84
NE = 0
NS = 0
NRJ= 13
NL = 4
XMAX= 972.0
YMAX= 770.4
ZMAX= 624.0



S T A A D P O S T - P L O T (REV: 22.3)

DATE: DEC 30, 1997

TITLE: - PORTLAND AIRPORT (AIRPORT11)

Attachments

UNEDITED SURFACE WEATHER OBSERVATIONS (METAR/SPEC)	LATITUDE	LONGITUDE	STATION ELEVATION FT. (MSL)	TIME CONVERSION	DAY	MONTH	YEAR	SID	STATION (TYPE, NAME, STATE)
	45°36'N	122°36'W	21	+8	31	07	1997	PDX	ASOS PORTLAND, OR

TYPE M/S 1	TIME (LST) 2	WIND DIR TRUE 3	SPD KTS 4	GUST KTS 5	VARIABILITY TRUE 6	VISIBILITY SURFACE STATUTE MILES 7	RUNWAY VISUAL RANGE (FEET) 8	PRESENT WEATHER 9	SKY CONDITION 10	TEMP °C 11	DEW POINT °C 12	ALTI-METER INS. 13	REMARKS AND SUPPLEMENTAL CODED DATA 14
M	0056	340	3			10			CLR	17	12	A3010	SLP193 T01670122 53006 (CR)
M	0156	340	4			10			CLR	16	12	A3010	SLP191 T01610122 (CR)
M	0256	340	4			10			CLR	15	12	A3010	SLP193 T01500122 (CR)
M	0356	320	3			10			CLR	15	13	A3011	SLP195 T01500128 10194 20150 53002 (CR)
M	0456	340	4			10			CLR	14	13	A3012	SLP199 T01440128 (CR)
M	0556	340	4			10			FEW010	14	13	A3013	SLP203 T01440128 (MD)
M	0656	360	3			10			SCT010	15	13	A3014	SLP207 T01500128 51011 (MD)
M	0756	000	0			10			SCT010	17	13	A3016	SLP211 T01670133 (MD)
M	0856	290	4			10			FEW010 SCT250	18	13	A3015	SLP211 T01780128 (MD)
M	0956	VRB	4			10			FEW020 SCT250	21	13	A3014	SLP207 T02060133 10206 20144 50002 (MD)
M	1056	VRB	4			10			FEW020 SCT250	22	13	A3013	SLP203 T02170128 (MD)
M	1156	300	9			10			FEW020 SCT250	23	12	A3012	SLP200 T02280122 (MD)
M	1256	VRB	5			10			FEW030 SCT250	24	12	A3012	SLP198 T02390122 56009 (MD)
M	1356	320	6			10			FEW050CB SCT250	25	12	A3010	SLP193 CB DSNT E T02500122 (JDS)
M	1456	310	6			10			FEW050CB SCT250	26	12	A3009	SLP188 CB DSNT E T02610122 (JDS)
M	1556	290	7			10			FEW050CB SCT250	27	13	A3008	SLP185 CB DSNT E-SE T02670128 10267 20200 5 6013 (JDS)
M	1656	310	7			10			FEW050CB SCT250	27	13	A3007	SLP183 CB DSNT ESE T02670133 (JDS)
M	1756	350	8			10			FEW055 SCT120 SCT250	26	13	A3008	SLP186 T02610128 (JDS)
									*** End of File ***				

UNEDITED SURFACE WEATHER OBSERVATIONS (METAR/SPEC)	LATITUDE	LONGITUDE	STATION ELEVATION FT. (MSL)	TIME CONVERSION	DAY	MONTH	YEAR	SID	STATION (TYPE, NAME, STATE)
	45°36'N	122°36'W	21	+8	31	07	1997	PDX	ASOS PORTLAND, OR

TIME LST 16	TOTAL SKY COVER (0–8) 17	TEMP. DRY-BULB 18	DEW POINT 19	TEMP. WET BULB 20	RELATIVE HUMIDITY (%) 21	STATION PRESSURE (INS) 22	PRESSURE TENDENCY 23	NET 3-HR CHANGE 24	HOURLY PRECIPITATION (INS) 25	HR
0056		16.7	12.2	142	75	30.08	3	006	0.00	00–01
0156		16.1	12.2	139	78	30.08			0.00	01–02
0256		15.0	12.2	135	83	30.08			0.00	02–03
0356		15.0	12.8	138	87	30.09	3	002	0.00	03–04
0456		14.4	12.8	135	90	30.10			0.00	04–05
0556		14.4	12.8	135	90	30.11			0.00	05–06
0656		15.0	12.8	138	87	30.12	1	011	0.00	06–07
0756		16.7	13.3	148	81	30.14			0.00	07–08
0856		17.8	12.8	150	73	30.13			0.00	08–09
0956		20.6	13.3	163	63	30.12	0	002	0.00	09–10
1056		21.7	12.8	165	57	30.11			0.00	10–11
1156		22.8	12.2	166	51	30.10			0.00	11–12
1256		23.9	12.2	170	48	30.10	6	009	0.00	12–13
1356		25.0	12.2	174	45	30.08			0.00	13–14
1456		26.1	12.2	178	42	30.07			0.00	14–15
1556		26.7	12.8	183	42	30.06	6	013	0.00	15–16
1656		26.7	13.3	186	44	30.05			0.00	16–17
1756		26.1	12.8	181	44	30.06			0.00	17–18

SYNOPTIC DATA

TIME 26	NO 27	LOW CLOUD TYPE 28	MID CLOUD TYPE 29	HIGH CLOUD TYPE 30	MAX TEMP (.1 C) 31	MIN TEMP (.1 C) 32	PRECIP (INS) 33	SNOW FALL (INS) 34	SNOW DEPTH (INS) 35	STATION PRESSURE (INS) 36	BAROGRAPH 37	BARR CORR 38	LOCAL USE 39

Synoptic Data Not Reported For ASOS Stations

WEATHER AND OBSTRUCTION TO VISION

TYPE 40	BEGIN 41	ENDED 42	TYPE 40	BEGIN 41	ENDED 42	TYPE 40	BEGIN 41	ENDED 42	TYPE 40	BEGIN 41	ENDED 42

No Wx Duration Data Reported

SUMMARY OF THE DAY (MIDNIGHT TO MIDNIGHT)

PEAK WINDS SPEED (MPH) 43	PEAK WINDS DIREC-TION 44	PEAK WINDS TIME (LST) 45	FASTEST 2-MIN WIND SPEED (MPH) 46	FASTEST 2-MIN WIND DIREC-TION 47	FASTEST 2-MIN WIND TIME (LST) 48	SUNRISE TIME (LST) 49	SUNSET TIME (LST) 50	TOTAL SUNSHINE (MIN) 51	PERCENT PSBL SUNSHINE 52	CHARACTER SUNRISE 53	CHARACTER SUNSET 54	SKY COVER SUNRISE TO SUNSET 55	SKY COVER MIDNIGHT TO MIDNIGHT 56	24-HR MAX TEMP. (0.1 C) 57	24-HR MIN TEMP. (0.1 C) 58	24-HR PRECIP. WATER EQUIV (INS) 59	24-HR SNOW-FALL UNMLTD 60	1200 UTC SNOW DEPTH (INS) 61	WATER EQUIV. (INS) 62	STATION PRESSURE 63	SEA LEVEL PRESSURE 64
20	330	1929	17	330	1930	453	1941	M	M					27.2	14.4	0.00		M			

REMARKS, NOTES AND MISCELLANOUS PHENOMENA 65 :

Daily Remarks Data not reported for ASOS Stations

TIME CHECK–CLOCK CORRECT TO THE NEAREST MINUTE AT: / / /

Nava Contracting & Engineering, Inc.
2732 S.W. Talbot Rd.. Portland. Oregon 97201
Ph: (503) 243-2840 , Fax: (503) 525-0778

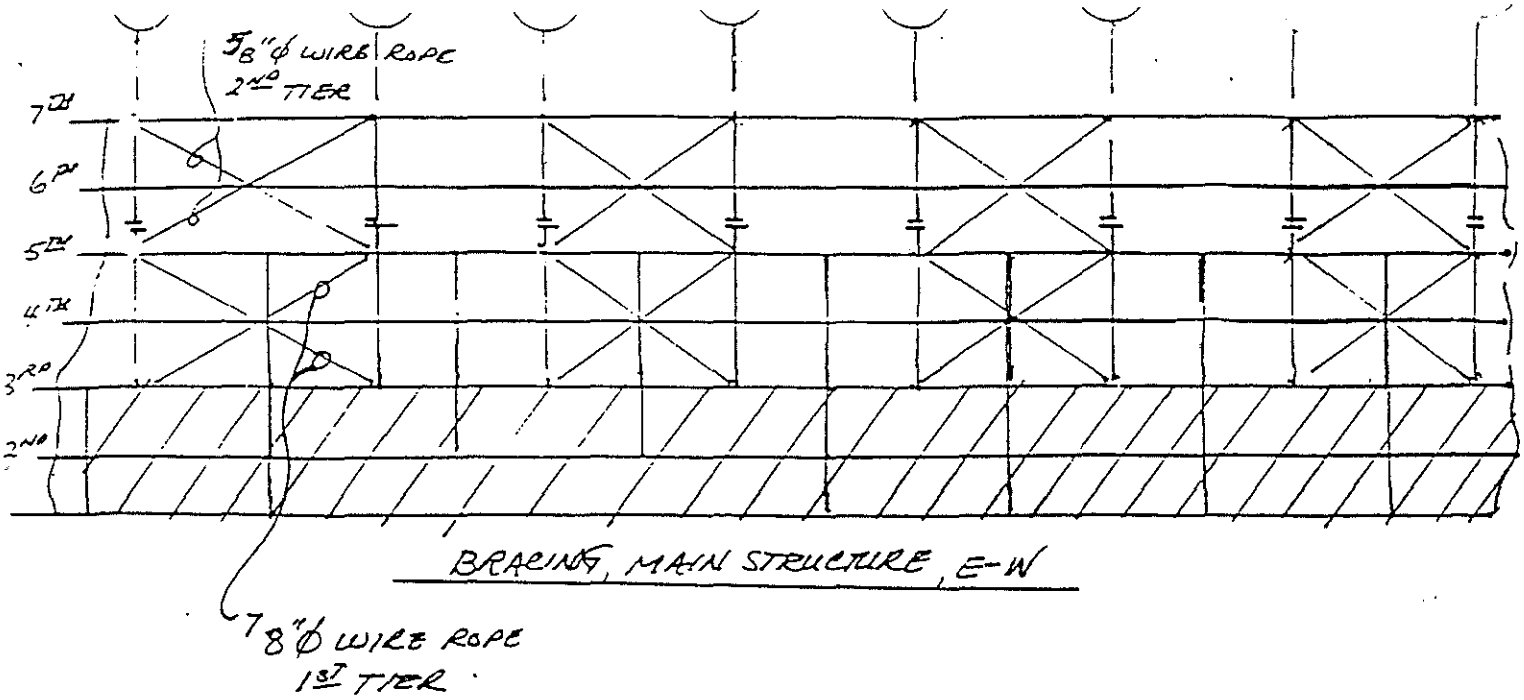
Erection Of Structural Steel Only (Cont.)

2. Erect structural steel between Grids P1 to P14. General erection sequence will be west to east (Grids P14 to P1). Following procedure is to be used:

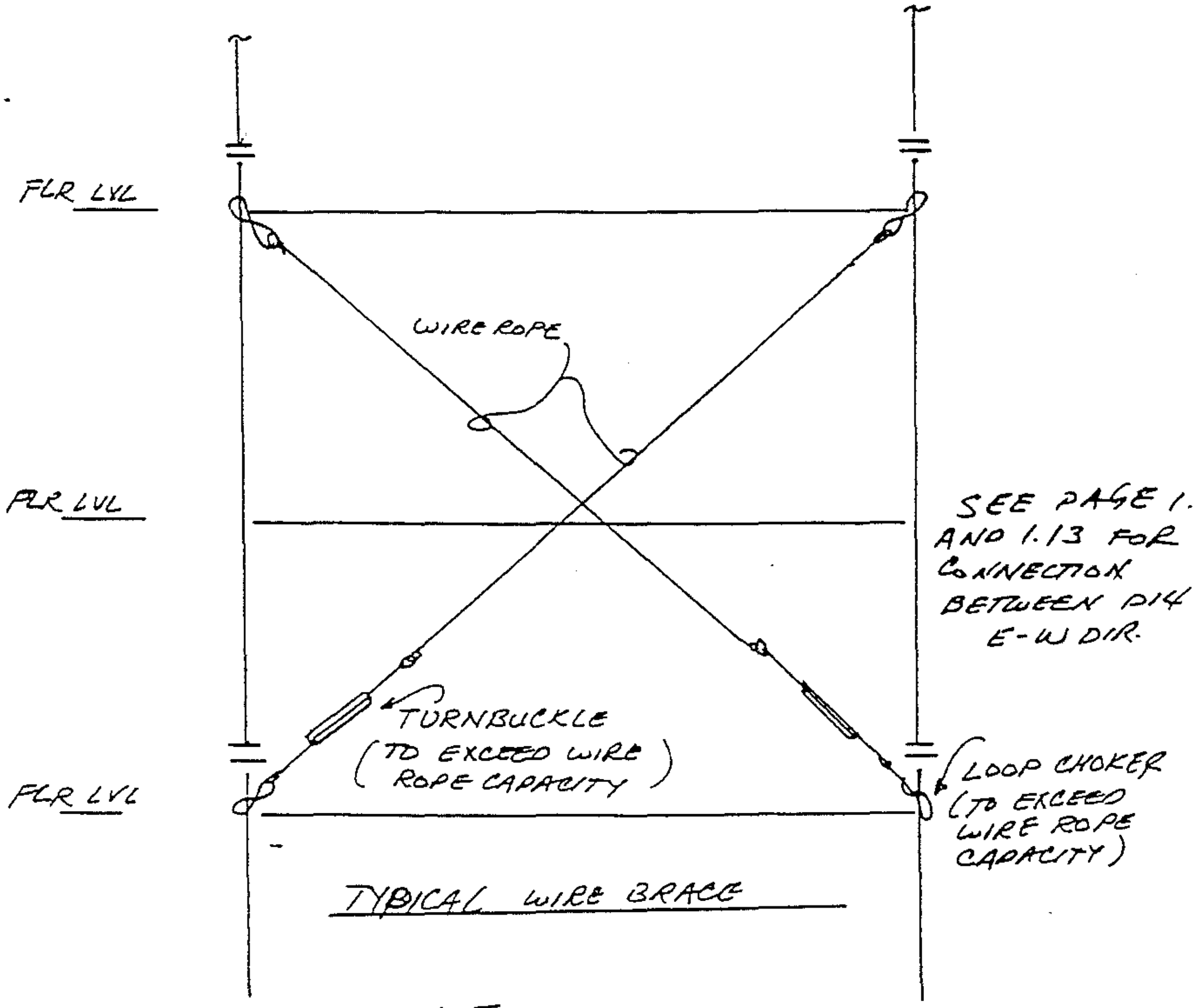
   a.) Erect first tier (first to third level) of columns, beams and bracing, and install tension bracing/guy wire at each bay in the east-west and north-south direction per the bracing plan provided.
   b.) Bolt, plumb, final torque bolts, and begin welding of moment connections. Bracing in N-S direction to be welded at one end and bolted w/ (2) 7/8 " diameter A325 bolts at other end. Bolted end to have bolts removed and brace connection welded after placement of concrete. Bracing on PB and PK to be welded as erected.

   c.) Erect second tier (third to fifth level) of columns, beams and bracing, and install tension bracing/guy wire at each bay in the east-west and north-south direction per the bracing plan provided.
   d.) Bolt, plumb, final torque bolts, and begin welding of moment connections. Bracing in N-S direction to be welded at one end and bolted w/ (2) 7/8 " diameter A325 bolts at other end. Bolted end to have bolts removed and brace connection welded after placement of concrete. Bracing on PB and PK to be welded as erected.
   e). Fully deck fifth level and begin decking fourth level. At a minimum, deck all perimeter bays will be decked : P12-P14, PK-PJ, PC-PB and P1-P3).

   f.) Erect final tier (fifth to seventh level) of columns, beams and bracing, and install tension bracing/guy wire at each bay in the east-west and north-south direction per the bracing plan provided.
   g.) Bolt, plumb, final torque bolts, and begin welding of moment connections. Bracing in N-S direction to be welded at one end and bolted w/ (2) 7/8 " diameter A325 bolts at other end. Bolted end to have bolts removed and brace connection welded after placement of concrete. Bracing on PB and PK to be welded as erected.
   h). Fully deck sixth and seventh level.



BRACING, MAIN STRUCTURE, E-W



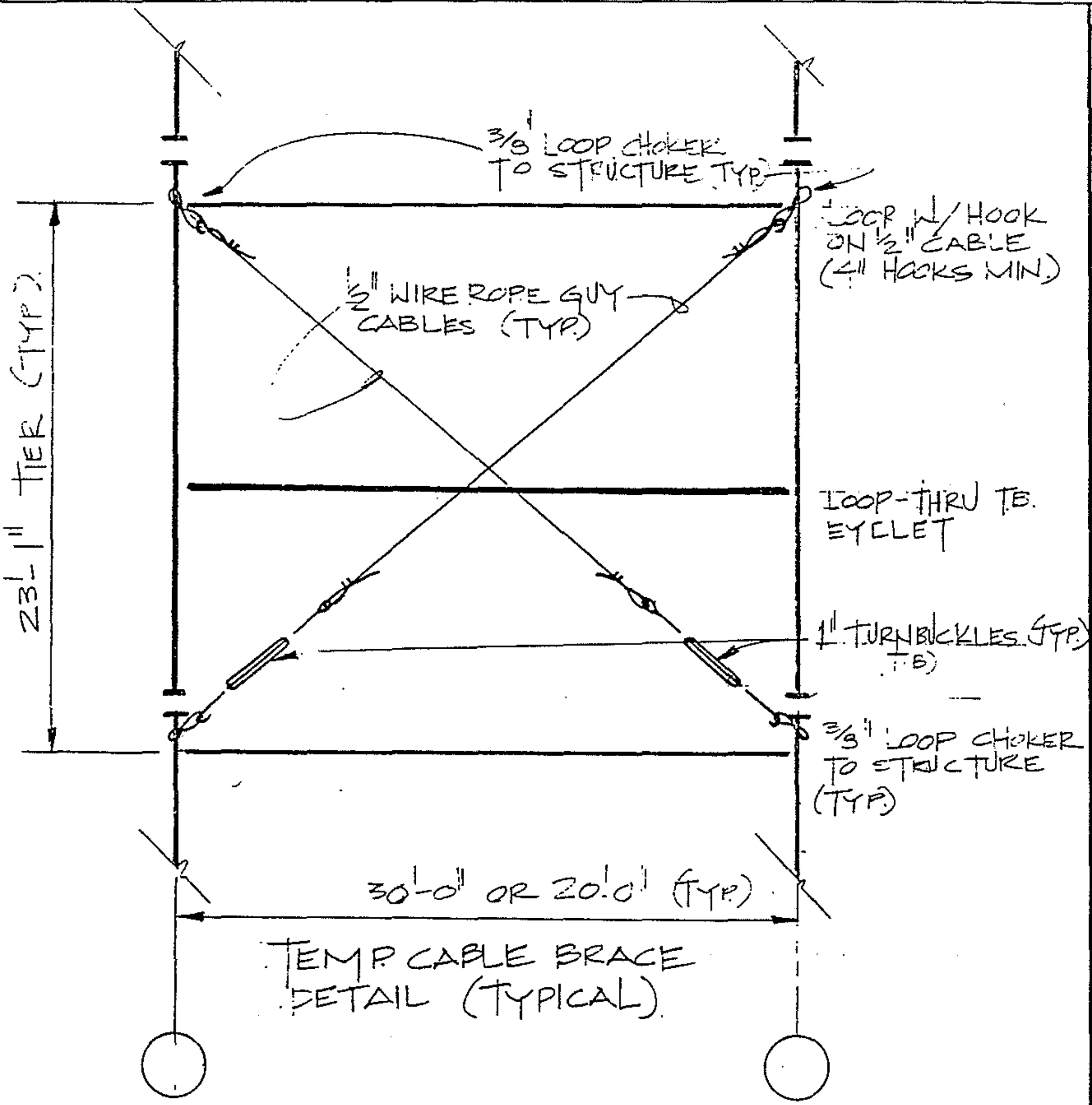
TYPICAL WIRE BRACE

P.O. Box 301158
Portland, OR 97294-9158
Phone (503) 254-5322
FAX (503) 254-5266

JOB No.	DESIGN by	DATE
PDX - 3030	[illegible]	7/10/9
SUPPORT	MAN HOURS	
	N/A	

CUSTOMER HAUGH

DESCRIPTION TYPICAL CABLE STAY/BRACE DETAIL



TEMP CABLE BRACE DETAIL (TYPICAL)

NOTES: 1) LOOP CHOKE = DOUBLE 3/8" WIRE ROPE WHEN CHOKING STRUCT.
2) 1/2" CABLE LOOPS TIED OFF W/ 2 - 1/2" CABLE CLAMPS (EA.) (CLAMP DEAD END.)
3) 1/2" CABLE TO BE DRAWN-UP TAUT.
4) ALL RIGGING EQUIPMENT / MATERIAL & PRACTISES - STANDARD TO IRON WORKER TRADE.
5) MIN. 2" THREAD SHOWING ON TURNBUCKLE.

8435 Baldwin Street
Oakland, CA 94621
Phone (510) 729-0620
FAX (510) 569-0377

9.4 During the final placing of solid web structural members, the load shall not be released from the hoisting line until the members are secured with not less than two bolts, or the equivalent, at each connection, to keep members from rolling and to sustain anticipated loads. Bolts shall be drawn up wrench-tight.

9.5 When double connections are involved, the structural detailer and fabricator shall be consulted concerning the provisions for a seat lug or flange length extension on one of the beams, and a corresponding bolt hole in the web of the column floor or beam.

9.6 If connecting lugs are bent, they shall be straightened before hoisting the member.

9.7 When columns are being set on base plate or shims, and before lifting falls are unhitched, either the nuts on the anchor bolts shall be drawn down tight or temporary guys shall be affixed.

9.8 A piece shall never be cut loose until the required minimum of bolts have been installed; a wrench or driftpin in the hole shall not be used as a substitute for the bolts.

10. Ladders

Ladders shall be constructed, used, inspected, and maintained in accordance with ANSI A14.1-1982, ANSI A14.2-1982, ANSI A14.4-1979, and ANSI A14.5-1982.

11. Structures

11.1 Buildings

11.1.1 Permanent floors shall be installed as soon as possible as the erection of structural steel members progresses. The sequence of erection, bolting, temporary guying, riveting, and welding shall be such as to maintain the stability of the structural frame at all times during the construction. Consideration shall be given to the dead weight of the structure, the weight and working reactions of all construction equipment placed thereon, and all external forces that may be applied.

11.1.2 There shall be a floor or safety net within two stories or 30 feet, whichever is less, below and directly under that portion of each tier of beams on which any work is being performed, except gathering and stacking temporary floor planks on a lower floor in preparation for transferring such planks to an upper working floor. The steel erector's personnel shall remove such planks successively, working toward the last panel of such floor so that the work is always done from the planked floor. When gathering and stacking temporary floor planks from the last panel, workers assigned to such work shall be protected by safety belts with lanyards attached to a catenary line or other substantial anchorage.

11.1.3 Where planking or safety nets would interfere with the erection of a structure, the use of safety belts with lanyards or other fall-protection systems shall be accepted as alternative protection.

11.1.4 In elevator shafts, a tight platform of planks at least 2 inches thick, full-size, or exterior-grade plywood at least 3/4 inch thick, or equivalent material, shall be installed not more than two stories or 30 feet, whichever is less, below the level at which people are working.

11.1.5 During the construction of a building, there shall be at least two stairways; or else a personnel hoist complying with the requirements of ANSI A10.4-1981 shall be installed to within four floors or 60 feet (whichever is less) of the uppermost working floor. Stairways shall not be located adjacent to each other, but shall be no more than 150 feet apart. A stairway may consist of steel scaffold with stairs.

11.1.6 Temporary stairways shall have treads constructed of wood planks not less than 2 × 10 inches in size, or of metal, not less than 10 inches in width, of equivalent strength. Such temporary stairways shall be not less than 3 feet in width, rigidly braced, and of sufficient strength to support a load of 100 pounds per square foot.

11.1.7 Stairways with steel treads, pan type stairs and landings without permanent surfaces, or both, shall be provided with adequate wooden treads and landings fitted securely in place to eliminate tripping hazards. Stairways that are not safe or ready for pedestrian traffic shall be blocked off to prevent usage.

11.1.8 Every stairway shall be provided with handrails of height not less than 30 inches or more than 34 inches, measured vertically from the nose of the tread to the top of the rail.

11.1.9 All structural steel framing shall have its structural integrity protected promptly upon erection by bracing adequate to resist horizontal forces such as wind and reactions by erection equipment.

11.1.10 No load-bearing structural member shall be materially weakened by cutting, holing, or other means, except with the approval of the designer of the structure or a licensed professional engineer.

11.1.11 Open-web steel joists (bar joists) placed at permanent or final positions shall be promptly bolted, riveted, or welded, and all permanent bridging shall be installed.